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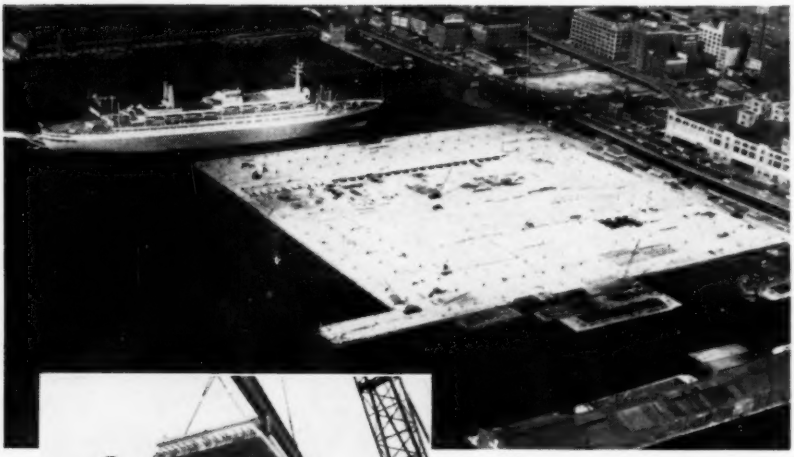
No. 5
V. 31

JOURNAL
OF THE AMERICAN CONCRETE INSTITUTE

ACI awards and the convention program appear in this month's News Letter

ACI PROCEEDINGS V. 56

15 ACRES OF P/S SLABS



(Above) New Flagship Rotterdam at Pier 40 on maiden voyage. Pier will have all-concrete superstructure. (Left) Typical slab being unloaded on pier deck. Most slabs measured 5' x 20' x 12"; weighed 7½ tons.

● For the new Holland-America Line Pier in New York, 6,021 prestressed concrete slabs were used in order to accommodate extremely heavy ultimate loads on a deck of minimum thickness (12") resting on widely-spaced stringers (20' apart). Design loads ran as high as 600 pounds psf.

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EDWARD K. RICE and FELIX KULKA open the February issue with an explanation of the "Design of Prestressed Lift Slabs for Deflection Control."

Coauthors ROGER DIAZ DE COSSIO and CHESTER P. SIESS present a research paper on "Behavior and Strength in Shear of Reinforced Concrete Beams and Frames Without Web Reinforcement."

HENRY TOENNIES describes research conducted to develop a process of "Artificial Carbonation of Concrete Masonry."

A rational analysis for determing the "Distribution of Torsion and Bending Moments in Beams and Slabs Connected Together" is presented by M. A. GOUDA.

Major factors influencing the "Shrinkage and Creep of Concrete" are set forth by INGE LYSE. Author includes formulas for calculating shrinkage and creep with a fair degree of accuracy.

Photographs courtesy Celso Carbonell

On the Cover—Apartment building in the La Cresta section, Panama City, Panama. Each floor of this all concrete structure is a separate apartment. Concrete was selected for its function and durability in this tropical climate. Penthouse is roofed with a hyperbolic paraboloid. Architect was Calvin E. Stempel; structural engineer was Diego E. Pardo, Jr.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

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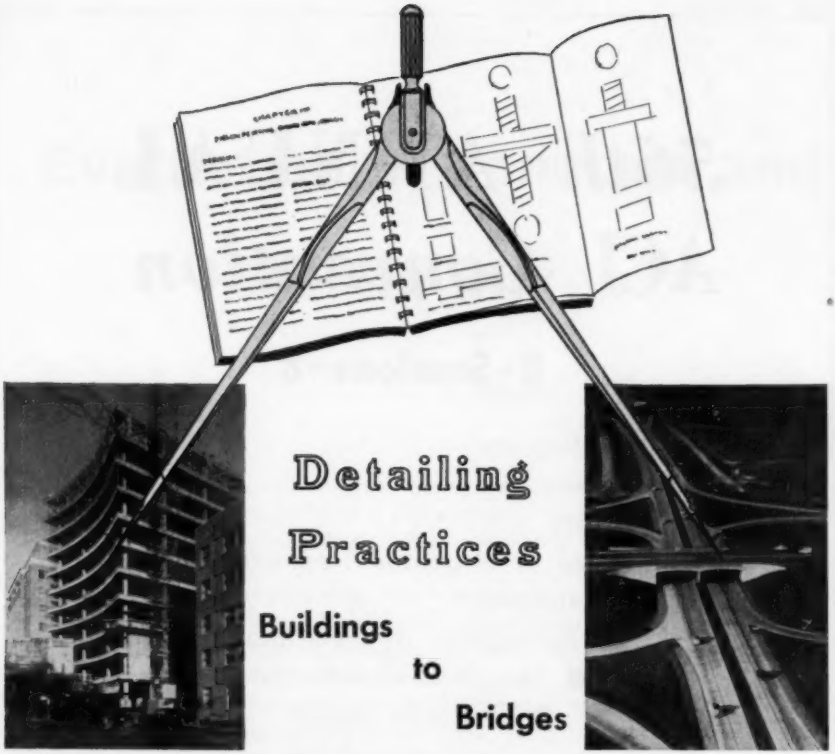

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Evaluation of Concrete and Mortar Mixes

By WILLIAM A. CORDON

Properly proportioned concrete mixes should be designed to produce (1) quality, (2) workability, and (3) economy. Required quality and adequate workability are mandatory, but to completely evaluate a mix, its cost should be considered.

Useful information regarding concrete can be obtained by evaluating concrete mortars. This is accomplished by evaluating the mortar from concrete mixes and correlating with concrete tests. New laboratory apparatus used in the study of concrete mortars is discussd.

Test results are included in which the following were evaluated: (a) aggregate grading, (b) aggregate particle shape, (c) portland cement, (d) pozzolans, (e) admixtures, and (f) water-cement ratio.

PERFORMANCE OF CONCRETE STRUCTURES is dependent on the quality of the concrete used. Concrete quality is generally established by its strength and durability, but in special cases permeability, volume change, thermal properties, and resistance to chemical attack may be important. Although quality is the most important characteristic of concrete, and to a large extent controls concrete proportioning, it is not the only characteristic which must be considered to properly evaluate a concrete mix.

The usefulness of concrete as a construction material is determined primarily by its (a) quality, (b) workability, and (c) economy. Concrete must be sufficiently workable to be properly placed or the potential quality of the concrete may be lost. It is also important to consider the comparative costs of concrete mixes.

Since the cost of concrete construction generally exceeds the cost of materials used, an "extra bag of cement" in concrete may be considered extra insurance of minor importance as far as the cost of the structure is concerned. For large massive structures, however, only a few cents difference in the cost of a cubic yard of concrete can amount to hundreds

WILLIAM A. CORDON, winner of ACI's Wason Medal in 1955 and member of ACI Committees 115, 116, 214, and 613, is presently in charge of the materials laboratory of Utah State University. Before accepting his present position he worked on performance tests for cement at the research laboratory of the Portland Cement Association. Prior to this, and for 14 years, he was engaged in investigating concrete and concrete aggregate for the U. S. Bureau of Reclamation's research laboratories at Denver.

of thousands of dollars. For centralized concrete plants which handle increasingly large volumes of concrete, small differences in the cost of a concrete mix may be important in the final margin-of-profit for the operation.

Concrete mortars consist of all the ingredients of a concrete mix with the exception of the coarse aggregate. A comparative evaluation of many of the characteristics of concrete may, therefore, be made using concrete mortars. Laboratory equipment has been developed which is sensitive to small changes in the consistency of concrete mortars.

Extensive investigations can be made and valuable test data obtained using mortar tests. Extensive concrete tests may not be practical when only small samples of material are available, laboratory facilities and personnel are limited, or funds are not available.

This paper proposes methods and laboratory equipment to evaluate concrete mortars. Test results with mortars are correlated with concrete tests and methods are suggested for presenting these results for evaluation of concrete and mortar mixes.

CONCRETE MORTARS

Concrete is composed of mortar with coarse aggregate added as an economical filler material. It is axiomatic that with a given coarse aggregate, the workability and quality of mortars will determine, to a large extent, the workability and quality of resulting concrete. It has been found that the grading, particle shape, and general characteristics of the sand fraction of aggregate may influence the workability of a concrete mix more than the grading, particle shape and general characteristics of the coarse aggregate.

Comparative tests on workability, water requirement, hardening characteristics, and quality of concrete mortars will provide valuable information regarding, (a) the influence of gradation and particle shape of sand; (b) the fineness, water requirement, and strength producing characteristics of cements and pozzolans; (c) the influence of admixtures; and (d) effect of temperature, mixing time, and other miscellaneous variables. Properties of concrete which cannot be evaluated with mortars include; (a) the influence of grading, particle shape, and quality of coarse aggregate; and (b) the percentage of mortar of a given workability required to produce concrete of a given slump.

Evaluating concrete mortars

The quality of concrete mortars may be determined by strength and durability tests in accordance with ASTM standards. The method for measuring the consistency and workability of mortars described in ASTM C 109-54, using the flow table described in ASTM C 230-55T, is not suitable to measure workability of concrete mortars, since a mortar in a concrete mix is much wetter than the mortars prescribed in the ASTM tests. It is necessary, therefore, to use a test method sensitive to changes in consistency of concrete mortars with a sufficiently wide range to include mortars found in concrete having slumps ranging from 0 to approximately 8 in.

A cone penetrometer, Fig. 1, has been developed in the laboratories of the Portland Cement Association and Utah State University. This device, which is a modification of the Vicat apparatus, shows promise in measuring workability and consistency of concrete mortars. The frame is raised 2 in. from the platform to provide room for the cone. The indicator scale is divided into millimeters and consists of a machinists scale mounted on the frame. The cone is made of stainless steel (or aluminum), 1⅝ in. in diameter at the large end, 3⅝ in. long, and blunted to a bullet point a distance ⅛ in. from the point, making a completed length of 3½ in. The cone is hollowed out and threaded to a stainless steel tube so that the total weight is 400 g when using a stainless steel cone and 200 g with the aluminum cone. The container used is the standard 400 ml measure designated in ASTM C 185.

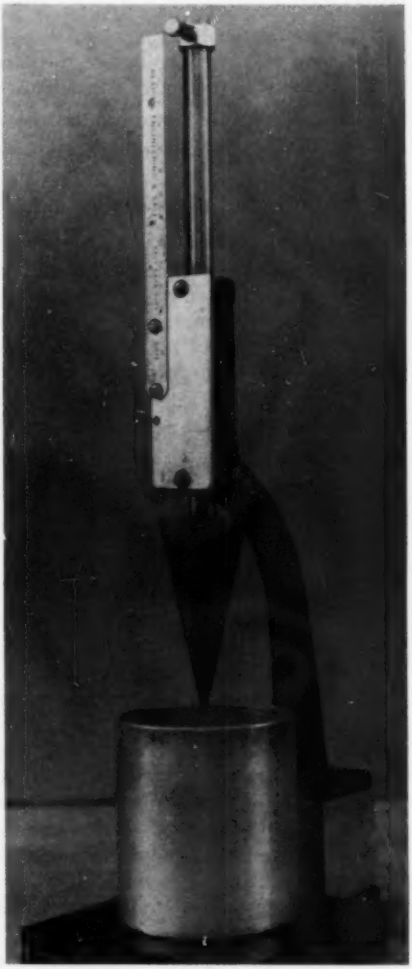

Fig. 1—Cone penetrometer for measuring consistency and workability of concrete mortars

Test procedure

The following testing procedure was found to be the most successful:

a. Compact the mortar in the container in 3 equal layers, tamping each layer 33 times in the following sequence. The first 5 tamps through the center of the container from one outside edge to the opposite side. Three additional tamps on each side of and parallel to the first 5 tamps. Rotate the container 60 deg and repeat. Rotate the container another 60 deg and complete the tamping.

b. Strike off container with a sawing motion using a sharp straightedge held perpendicular.

c. Raise the penetration cone and slide container underneath until the point of the cone rests on the edge of the container. Tighten the set screw just enough to hold the cone and move the zero point of the scale opposite the indicator.

d. Center the container under the cone and release the cone with a swift definite turn of the set screw while holding the container firmly with the left hand.

e. Record depth of penetration in millimeters.

Laboratory tests on mortar, screened from a concrete mix having 1-in. slump, showed a penetration of approximately 50 mm when using the 200 g aluminum cone while mortar from concrete having a 6-in. slump showed a penetration of approximately 80 mm. For standard mortars used in testing cements, ASTM C 109-54, the heavier stainless steel cone gives a better range when evaluating the consistency of these stiff mortars.

Explanation of the use of terms

Although other important properties, in addition to strength, determine the quality of concrete, for the sake of simplification in this paper quality will be measured by the strengths of mortar cubes and concrete cylinders. Strength, durability, and other desirable characteristics of concrete vary with the water-cement ratio, however, and therefore correlate with each other for particular materials. A more complete presentation of the proposed method of evaluation would include curves showing durability factors treated in much the same way as strength.

Workability of mortar and concrete is measured by penetration and slump tests. These tests do not measure all of the characteristics of workability, and may not measure the same properties of workability. Each varies with the consistency, however, which for practical purposes is generally considered synonymous with workability for any particular concrete. Consistency and quality vary with the water content for a given amount of cement and hence provide correlation between workability and quality.

Cost of concrete is determined by the bags of cement used. The "efficiency" of a concrete mix on a strength-cost basis is the amount of strength produced for a given amount of cement and is expressed as psi per pound of cement per cubic yard of concrete or psi strength per gram of cement per liter of mortar. The efficiency computed for either concrete or mortar is about the same magnitude. In the same manner durability efficiency could be expressed as cycles of freezing and thawing to failure per pound of cement per cubic yard of concrete. Tests are not yet available to demonstrate durability efficiency.

MORTAR TESTS

The following results of laboratory investigations illustrate the use of the penetrometer in evaluating influence of sand grading, different sands, cements, pozzolans, and admixtures on the properties of mortars.

Grading of sand

To provide a wide variation in sand grading, a commercial sand was separated into various seive sizes and recombined to produce fineness moduli ranging from 2.00 to 3.02. Fig. 2 shows the results of tests made with these sands when mixed in mortars. The lower portion of Fig. 2 shows the depth of penetration of the cone penetrometer at different water contents for mixes made with each sand. The upper part of Fig. 2 shows the comparative strength of mortars made with different sand gradings and different water contents. At a penetration of 60 mm, the water requirement of the several sands varies from 268 to approximately 306 mm per liter of mortar. This corroborates previous findings which show that the water requirement of mortar mixes decreases as the grading of sand becomes coarser. The strength tests show the greater strength producing properties of properly graded sand with lower strength for very fine and very coarse sands.

Test data plotted in the above manner provide a convenient method for evaluating various properties of mortar mixes. By moving horizontally the various sands may be evaluated for strength and water requirement at the same workability. By moving vertically the same data can

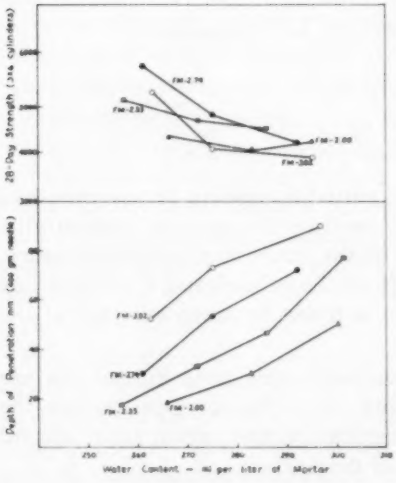

Fig. 2—Evaluation of the influence of sand gradings in mortars

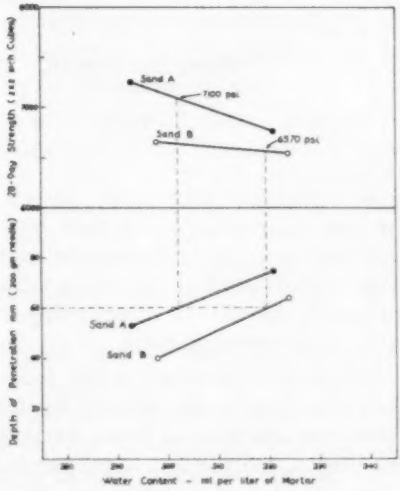
28-Day Strength (4x8 inch Cubes)
Sand A
7100 psi
Sand B
6570 psi
Depth of Penetration mm (300 gm needle)
Water Content – ml per liter of Mortar

Fig. 3—Evaluation of two commercial sands in concrete mortars

be used to evaluate workability and strength for the same water-cement ratio. A constant cement content was used in all mixes of each series.

Evaluation of two commercial sands

Test results shown in Fig. 3 are used in evaluating two commercial sands. Such an evaluation may be made to check the comparative value of available materials and shows water requirement and strength producing ability without testing grading, particle shape, or other physical characteristics. Fig. 3 indicates that Sand A requires 302 ml of water per liter of mortar while the Sand B requires 319 ml or about a 6 percent increase. The comparative efficiency of these two sands in producing strength either at equal consistency or equal water-cement ratio, is also indicated in Fig. 3. At a 60 mm penetration and equal cement contents Sand A produced 7100 psi while Sand B produced only 6570 psi.

Cements and pozzolans

Two important criteria used in evaluating cements and pozzolanic materials is their water requirement and ability to produce strength. Fig. 4 and 5 illustrate how the proposed method of evaluation can be used to determine these characteristics for cements and pozzolanic materials.

Fig. 4 shows the relationship between the workability, the water requirement, and the strength of mortars containing two Type I cements. From these tests an accurate quantitative comparison can be made of the water requirement and strength at the same (60 mm) penetration.

	Cement A	Cement B
Water requirement, ml per l	301	317
Strength, psi	3600	2900

As mentioned previously one of the valuable aspects of the proposed method of evaluation is that by intersecting the curves horizontally, comparison is made between cements at the same workability (penetration), as indicated in the above table. If on the other hand a vertical line intersects the data, a direct comparison is made between cements at the same water-cement ratio.

Test results obtained when four pozzolans were used in mortars are shown in Fig. 5 (25 percent by weight). At a 60 mm penetration the water requirement of these various pozzolan mixes varied from 258 ml per liter of mortar to 270 ml per liter of mortar.

The comparative efficiency of each of these materials in producing strength after 180 days moist curing is also indicated in the upper portion

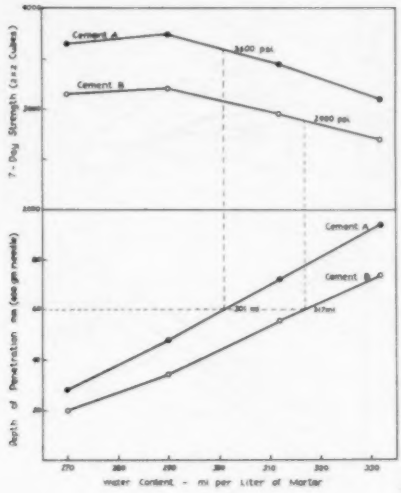

Fig. 4—Evaluation of two Type I cements with standard graded sand

of Fig. 5. The pozzolans may also be evaluated on the basis of either a given consistency or a given water-cement plus pozzolan ratio.

Admixtures in concrete mortars

Evaluation of admixtures used in concrete may be complicated, depending on the air-entraining properties of the admixtures, since all the characteristics of concrete, including the volume of mix, may be changed when admixtures are used. Comparative efficiency of concrete mortars provides a valuable tool for quantitative evaluation since the basic characteristics of quality, workability, and economy are considered for an equal volume of mortar rather than equal mix proportions.

Test results of mortar mixes in which air-entraining agents and water-reducing agents were used are illustrated in Fig. 6. These results show the water reduction accomplished when Admixture A, a water-reducing agent, is used and also the further reduction in water when Agent B, an air-entraining agent, is also used. The 7 day strength tests indicate higher strength at lower water contents (water-cement ratio) and also illustrate the effect of the air-entrainment in reducing strength at a given water-cement ratio.

Strength efficiency (psi per g per l), plotted on the ordinate at the right, are also shown in Fig. 6. This illustrates that the efficiency of air-entrained concrete in producing strength may not be decreased, as indicated by the solid lines, when the volume of batch produced is considered. In other words, the cement in the air-entrained mortar must provide strength for a larger volume of mortar. The strength of the larger volume of air-entrained mortar is decreased as shown by the solid

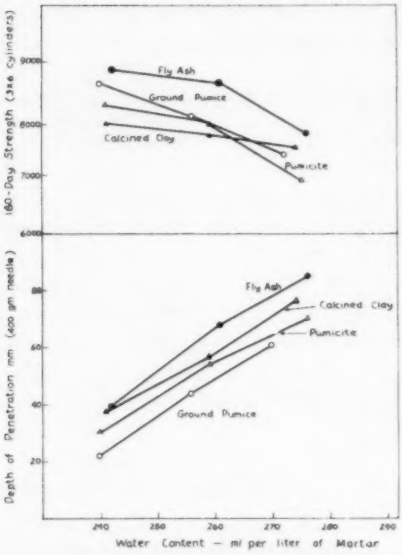

Fig. 5—Evaluation of pozzolans in concrete mortars

lines, but the "strength efficiency" is not decreased as shown by the dashed lines.

CONCRETE MIXES

Concrete mixes can be evaluated much in the same manner as concrete mortars. Tests for strength and durability are well established as methods of determining concrete quality. The most realistic procedure for evaluating quality is measuring the actual performance of field materials under standard laboratory conditions.

To obtain the greatest value from tests of concrete mortars, it is desirable to correlate these tests with tests of concrete. The slump of concrete is used as the measure of consistency and workability of concrete. Fig. 7 gives an indication of the relationship between depth of penetration of mortars and the slump of concrete made with these mortars. This direct correlation was accomplished by wet-screening mortar from a concrete mix of known slump and immediately measuring the depth of penetration.

The accuracy of measuring slump in concrete (0 to 6 in.) is not as great as the accuracy of measuring penetration in mortars (0 to 90 mm). The slump test was not found to be as accurate in measuring small changes in consistency and workability. This would indicate that for some investigations mortar tests may be superior to concrete tests.

The following examples show how proposed methods may be used to evaluate various characteristics of concrete mixes.

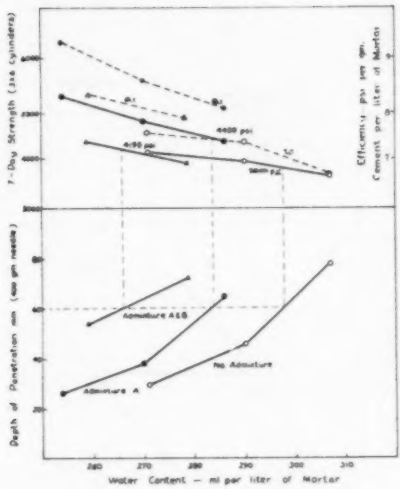

Fig. 6—Evaluation of admixtures in concrete mortars. Ordinates on left are for solid lines and ordinate on right is for dashed lines

Particle shape of coarse aggregate

Particle shape of concrete aggregate has been a topic of much discussion. It is agreed that rounded aggregates fit closer together, leave less voids, and that less mortar is required to produce a given workability. It is contended that the shape of angular aggregates contributes to strength of concrete because of the increased bond and shearing strength.

Two aggregates were evaluated on the basis of equal workability (slump) and equal water-cement ratios. Laboratory tests shown in Fig. 8 were made with identical concrete mixes, with the exception that the coarse aggregate was hand picked. One sample contained 100 percent well rounded pebbles while the other contained 100 percent crushed angular material. A constant cement content was used, but the water content was varied to produce slumps from approximately 1 to 6 in. The results of these tests indicate that for a 4 in. slump, rounded aggregate required approximately 312 lb of water per cu yd, whereas the angular aggregate required approximately 342 lb of water per cu yd.

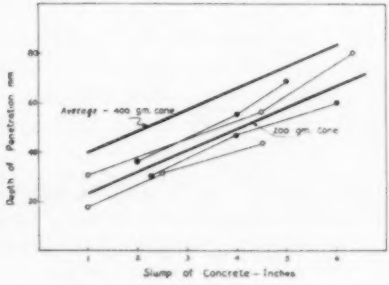

Fig. 7—Correlation between slump of concrete and depth of penetration. Mortar wet screened from the concrete mix

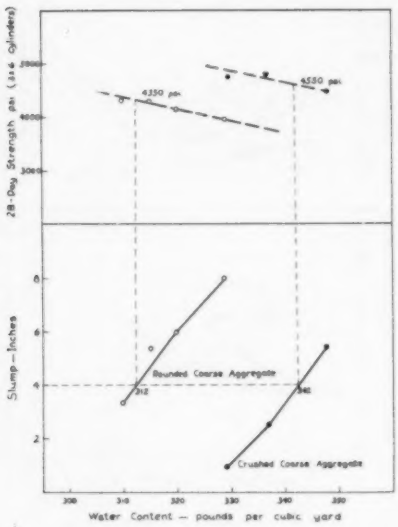

Fig. 8 — Evaluation of crushed and rounded coarse aggregate in concrete mixes—constant cement content

Without analyzing this information further, it could be assumed that at the higher water-cement ratio, the crushed aggregates produced an inferior concrete. In the upper portion of Fig. 10, the compressive strength of each mix is also plotted against the water content. Presented in this manner, the tests demonstrate that at a given water content, (water-cement ratio) the crushed aggregates produce higher strength concrete, of a much lower slump. For concretes of equal work-

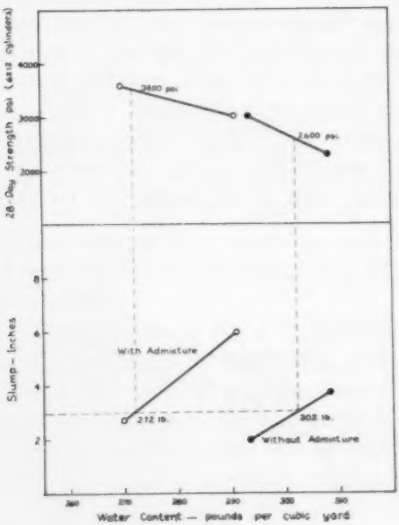

Fig. 9 — Evaluation of concrete mixes containing water reducing admixtures—constant cement content

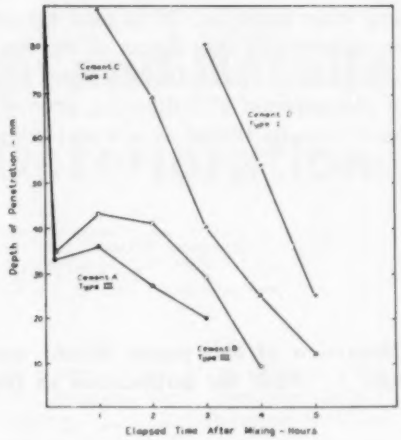

Fig. 10—Retention of plasticity of portland cement pastes with 50 percent water: Cements A, B, and C are known to have false setting tendencies. Cement D is a Type I cement in current production (courtesy of the Portland Cement Association)

ability, (4-in. slump) there is little difference in the compressive strength of concrete made with rounded and crushed angular aggregates. This would corroborate the contention that at a given cement content and consistency, crushed and rounded aggregates produce comparable strengths. The higher water-cement ratio (lower quality paste) required for crushed aggregates, to produce a given slump, should not be ignored. To completely evaluate these aggregates, durability tests as well as strength tests should be considered.

Admixtures

Tests made to evaluate the effect of water-reducing admixtures in concrete are illustrated in Fig. 9. For a slump of 3 in. a reduction in water from 302 lb per cu yd to 272 lb per cu yd, is indicated. At this consistency an increase in strength from 2600 psi to 3500 psi is indicated by using the admixture. Moving vertically at the same water-cement ratio, the mixes with and without admixtures indicate comparable strength, but about 4 in. difference in slump.

Premature stiffening of portland cement pastes

Test results (Fig. 10) are not included as part of the discussion on evaluation of concrete and mortar mixes, but show another possible application for the cone penetrometer. In this case, portland cement pastes containing 50 percent water by weight of cement were checked for depth of penetration at periodic intervals.

Pastes made with cement known to have false setting characteristics showed an immediate loss of penetration. Loss of penetration, therefore, provides a quantitative measure of premature stiffening tendencies at

any time interval. It is also interesting that there is a distinct difference between the shape of curves produced by Cements A, B, and C, although all have false setting tendencies. One superior consideration of this method of indicating premature stiffening is the fact that cement paste can be tested at a water-cement ratio commonly used in concrete mixes.

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Title No. 56-35

Effect of Design and Details on Concrete Deterioration

By P. D. MIESENHELDER

Features of design or of construction of a concrete structure often are important contributing causes of concrete deterioration. Primarily, deterioration is a result of freezing and thawing. Such deterioration is often thought of as a consequence of the number of times freezing and thawing occurs, but in the examples pictured the major factor is the high degree of saturation which existed at the time of freezing. This high degree of saturation is usually a consequence of inadequate or no drainage provisions at critical points. This paper consists essentially of pictures of structures from a large area which illustrate examples and the wide extent of occurrence.

THE DESIGN OF A CONCRETE STRUCTURE often establishes conditions which result in deterioration of the concrete. When deterioration is observed, naturally it is considered a weakness or shortcoming of the material. But this conclusion is confusing when it is observed that other portions of the same structure are in excellent condition and especially when it is obvious that the two portions were of the same materials and placed at the same time. As more and more structures are examined, even if built in different localities, of different materials, and in different years, it becomes apparent that there is a similarity of conditions existing where deterioration is evident. Primarily the deterioration is the result of freezing and thawing. The difference between the affected and unaffected parts results from the amount of water absorbed and the difference in absorption results from features of the structure. Where water is continuously available to the concrete for too long a time, such as results from inadequate or improper drainage, high absorption occurs.

FACTORS AND CONDITIONS

To understand why such conditions account for the difference between affected and unaffected parts, it is only necessary to remember certain established facts. It is recognized that when liquid water changes to ice an expansion occurs and if confined the force developed by the expansion is of a magnitude much greater than the strength of most ma-

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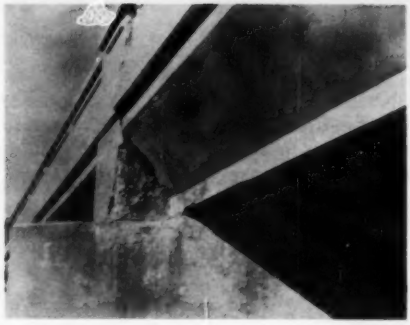
terials. For such force to be developed, a space or cavity must be filled or nearly filled[1] with water. For concrete the degree to which space or voids are filled with water is dependent on conditions which have prevailed. It is the prevailing conditions which are important in accounting for what occurs.

Perhaps various artificial conditions might be arranged which would produce a high degree of saturation. For example, vacuum saturation can so completely fill the void space with water that damage to the concrete occurs with only a few exposures to freezing.[2] But natural, not artificial, conditions are involved in the examples to be illustrated. The degree of saturation which contributed to deterioration results from continued contact with water. As a measure of absorption of concrete, standard test methods usually determine the amount of water absorbed during 24 hr of immersion, but that is by no means all of the water that can be absorbed. If the length of time of immersion is increased, so is the amount of absorption increased even though it may be at a decreasing rate. This is true even for dense materials.[3] The water which is absorbed during longer exposure may be extremely small in quantity, but critical when the concrete is exposed to freezing. This is because the voids may be filled beyond that point where there is sufficient space remaining to allow for the volume increase which occurs when the water is converted to ice. Confined, the force exerted by the expanding ice will rupture the concrete. This explanation may appear so simple that it seems elementary, but it often accounts for the deterioration which has occurred.

The feature which is common in all such cases is the presence of water for periods of time sufficient to produce an almost saturated condition of the concrete. Such conditions result from the design or construction and might be summed up as inadequate or improper drainage. A level surface, and not necessarily of extensive area, can cause trouble if there is a slow discharge of water onto it. The discharge may be either a slow continuous flow or an intermittent flow. Vertical construction joints and expansion joints may admit water more readily than they permit the escape of the water, in which case conditions favorable to high absorption are provided. In such cases there is little or no loss of water by evaporation and whatever water gains entrance is accumulated, thereby maintaining a wet condition for absorption during a longer time.



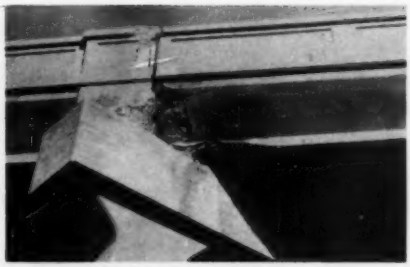


Fig. 1 (upper left)—Over-all view of grade separation

Fig. 2 (above)—Top of left-hand bent in Fig. 1. Water entered joint in deck, continuing between diaphragms

Fig. 3 (left)—Top of the second bent in Fig. 1. Result of water in joint

Conditions which result in a slow or intermittent flow of water are more common than seems to be generally recognized. One example would be a ballasted deck railroad bridge. Water enters the ballast readily, but it may drain from it slowly. Wet areas at the low or discharge point can be found for days after a rain. Another example, and one not restricted to the service of the structure, is water released from the melting of snow or ice. Melting may occur slowly during the daylight hours for several days and on level surfaces a thin layer of water is maintained during an extended time permitting a high degree of absorption. Should the surface drain into a crack or joint, as often happens, there are provided conditions for high absorption. If there is no outlet to the joint or only a restricted outlet the water may accumulate until a small hydraulic head exists.

These remarks are only preliminary to photographs of some bridges illustrating the conditions described. The convincing evidence is that the location and the type of deterioration is often repeated. Included in the examples are bridges as far east as Pennsylvania and as far west as Wyoming and Montana with a majority being in Indiana and Illinois.

DETERIORATION EXAMPLES

Fig. 1 is a general view of the first structure to be examined. Close-ups of two of the four bents are shown in Fig. 2 and 3. There is a joint through the deck slab which continues between the diaphragms at the ends of the beams. Water entering the joint reaches the bent cap where freezing and thawing deterioration occurs because of the

Fig. 4—Deterioration of cap and beam as a result of water in joint

Fig. 5—Deterioration of cap and beams. Bituminous material as applied would accelerate rather than retard deterioration

Fig. 6—Deterioration of cap and beams as a result of water in joint

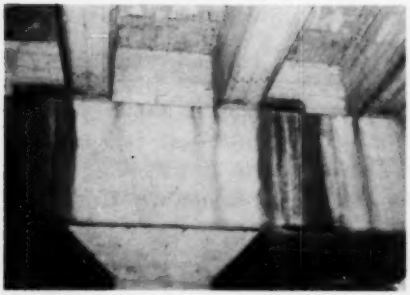


Fig. 7—Construction similar to that in Fig. 6 but openings in alternate diaphragms permitted escape of water. Though watermarked, concrete is sound

Fig. 8—Concrete deck on steel beams. Water entering joint at right contributed to deterioration

high moisture content in the concrete. Much of the spalling to be observed is due to causes other than freezing and thawing. The extent of the freezing damage is indicated by the discolored cracks.

A different structure of similar construction is seen in Fig. 4. The The bottom of two beams show deterioration near the cap and the effect of water and freezing is evident on the cap. Another structure of the same type is shown in Fig. 5. Deterioration is evident in the diaphragms, on the bottom of the beams, and on the cap. The bituminous material, as applied, would not prevent the entrance of water into the concrete, but would retard the escape of water by sealing the surface. This would tend to accumulate and increase the moisture in the concrete and hence would accelerate rather than retard the deterioration.

Fig. 9 — Deterioration in bent occurs where water came from above

Clearly evident in Fig. 6 is deterioration in the diaphragms, beams, and the cap which results from water in the joint back of the diaphragms and above the cap. In Fig. 7 a similar construction is seen except that openings were formed in alternate diaphragms which permitted the escape of water. Though the cap is discolored by water which drained through the openings, the concrete is sound.

In Fig. 8 a concrete deck is supported on steel beams. At the right, above the pier, can be seen an expansion joint. Deterioration caused by the freezing water which had been absorbed from the joint can be seen in the bottom of the slab.

Fig. 9 and 10 are two bents of the same structure. In Fig. 9 leakage is clearly evident and cracks which have developed are at or below the bearing where water escapes from the joint above. At the upper right

Fig. 10—Same structure as Fig. 9. Note that upper portion of column which was free of water from joint is sound

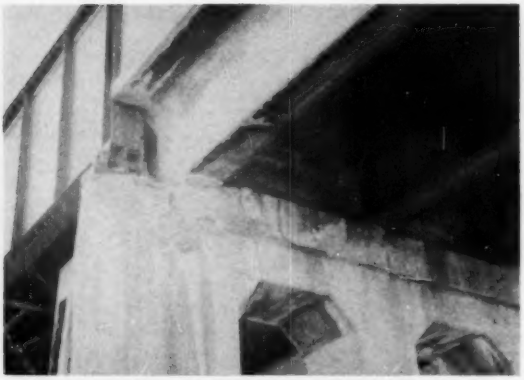


Fig. 11—Water from joint between spans and near curb line contributed to deterioration

of the photo (Fig. 9), notice that there is an air space between the top portion of the column and the outside beam. In Fig. 10 there is evidence of leakage and subsequent deterioration, but notice the upper part of the column above the bearing. Because of the air space and the handrail post above, this concrete was not wetted by leakage and is undamaged.

Fig. 11 is a bent supporting one end of a steel span and a concrete span. The roadway above drains to the sides and at least half of the steel span drains toward the concrete span. The joint between the spans has permitted water to drain onto the supporting concrete with the result shown in the picture.

The column in Fig. 12 supports the upper level of a two-level railway station. Damage occurred to the column as a result of leakage

Fig. 12 — Deterioration from leaking water before effective collection system was installed

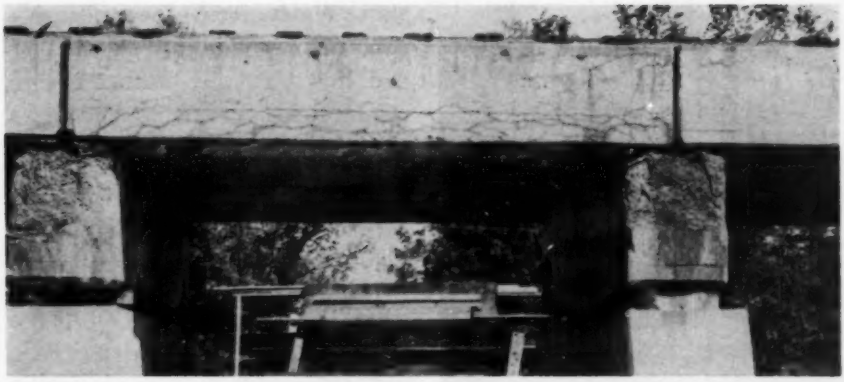


Fig. 13—Deterioration of caps below joints between precast slabs (reconstruction in progress)

from above and subsequent freezing before the installation of a collection system.

Fig. 13 shows deterioration, resulting from freezing and thawing, on the ends of the caps which support precast slabs of a railway trestle. Water draining from the ballast escapes through the transverse joint between the slabs contributing to this condition. (Reconstruction was in progress when this picture was taken.) There were two precast slabs in each span resulting in a longitudinal joint between them. Fig. 14 is a view of the underside of this joint. Here again water draining from the ballast produces high moisture content in the concrete with damage resulting when freezing occurred.

Fig. 15 is a pier of a railway precast slab bridge. Water draining from the ballast has found its way through the transverse joint above the pier. The resulting high absorption followed by freezing damaged the concrete.

An over-all view of a steel girder span with precast concrete slab approach spans is shown in Fig. 16. Fig. 17, taken from the left of the bridge, shows deterioration of the pier cap and the edge of the precast

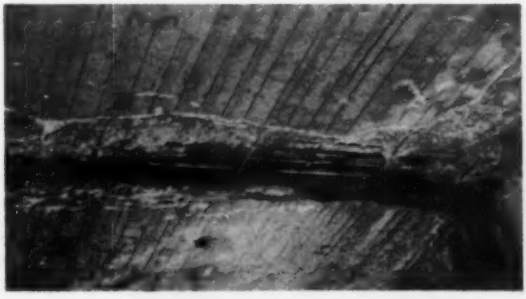


Fig. 14 — Deterioration on bottom of precast slabs at longitudinal joint from leakage through joint

Fig. 15—Pier deterioration below transverse joint between precast slabs

slab. Water from the track has drained over the curb of the slab and this, combined with freezing, produced conditions as shown. A similar situation occurs at the other end of the bridge.

Piers of a structure carrying a highway over a highway are shown in Fig. 18. Here again leakage at the joint over the pier, especially near the curb line, contributes to deterioration that occurs below.

Fig. 19 shows an abutment of a steel girder span. Unfortunately adequate provision was not made for all the drainage from the deck. Some leaked down onto the bridge seat with the result that there was produced a high moisture content in the concrete and with freezing the condition in the picture resulted.

LESSONS TO BE LEARNED

A question might be asked—was air-entrained concrete used in any of these structures? For one reason or another, it is fairly certain that none of them were built with air-entrained concrete. It would be conservative to believe that had concrete with proper air entrainment been used the deterioration observed would at least be much less, in some instances perhaps nonexistent. But to correct the basic cause of the trouble, adequate drainage or waterproofing should be provided.

The purpose of this paper was to illustrate and point out the cause

Fig. 16—Over-all view of structure with precast approach spans

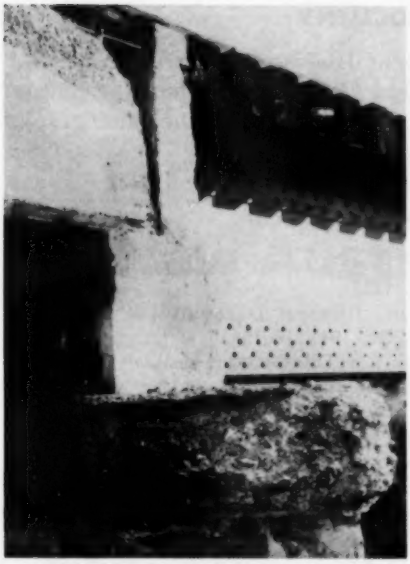


Fig. 17—View from left of pier in Fig. 16

Fig. 18—Damage as a result of water in transverse joint

of instances of deterioration which are all too common. An extremely high moisture content in the concrete contributes to damage when exposed to freezing and such high moisture content often results from lack of or inadequate provisions for drainage. Preventative measures would include avoiding designs that would pond or retain water, providing sloping surfaces which will drain readily, and the use of waterproofing or adequate sealing where needed.

It is hoped that this presentation may afford more general recognition of a condition which contributes to concrete deterioration and thereby be helpful to others in avoiding such conditions in future construction.

Fig. 19—Deterioration of bridge seat caused by drainage from structure

ACKNOWLEDGMENT

Acknowledgment is made to the Joint Highway Research Project of Purdue University and to C. C. Oleson, Portland Cement Association, for their courtesy in permitting the use of several of the photographs which have been included.

REFERENCES

1. Powers, T. C., "Basic Considerations Pertaining to Freezing-and-Thawing Test," *Proceedings*, ASTM, V. 55, 1955, p. 1135.

2. Withey, M. O., "Progress Report, Committee on Durability of Concrete," *Proceedings*, Highway Research Board, V. 24, 1944, p. 199.

3. Verbeck, George J., and Hass, Werner E., "Dilatometer Method for Determination of Thermal Coefficient of Expansion of Fine and Coarse Aggregate," *Proceedings*, Highway Research Board, V. 30, 1950, p. 192.

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Title No. 56-36

Design of Beams Subject to Torsion Related to the New Australian Code

By HENRY J. COWAN

Formulas, accompanied by examples, are presented for determining torsional shear stresses, diagonal tensile stresses, and angle of rotation for plain rectangular and circular beams, T-, L-, and I-beams, with flanges, and for rectangular box sections. (Most of these equations are included in the new Australian building code.)

Examples and formulas are also given for determining the amount of torsion reinforcement required for both circular and rectangular beams. The relative qualifications and limitations of these formulas are discussed. The final part of the paper discusses maximum permissible stresses in torsion, combined torsion and shear, and combined torsion and bending. An annotated bibliography on the torsional strength of concrete is appended.

TORSION IS ALMOST INVARIABLY a secondary effect in reinforced concrete buildings, and the subject has not therefore received the same attention as bending, transverse shear, or compression. Nevertheless, at least 12 experimental investigations are on record[1-13] and some involved the testing of a large number of full-size specimens. There is now sufficient information available to design concrete beams subject to torsion with due regard to both safety and economy, and the new Australian concrete code* includes, in Rule 615, provisions for the design of sections subject to torsion.

Torsion results essentially from the monolithic character of concrete construction. Thus any asymmetry in the loading of a floor slab produces torsion in the supporting beams. The twisting moments may exercise a controlling influence over the design of major edge beams, as in the case of the Waterloo Bridge[26] and the Royal Festival Hall balcony.[14]

*Australian Standard Rules for the Use of Normal Reinforced Concrete in Buildings, Standard Association of Australia, Sydney, 1958, 109 pp.

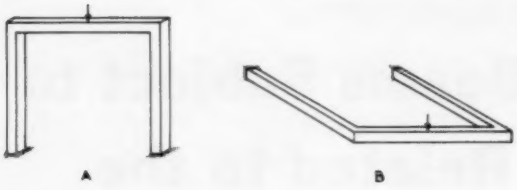

Fig. 1 — Rectangular frame used in a vertical and in a horizontal position

Frame A, used as a portal frame, is subject to combined bending and compression, while Frame B, used as a balcony beam, is subject to combined bending and torsion

Perhaps more important is the girder with a horizontal projection. The frame shown in Fig. 1 would, if used in the vertical position, be subject to combined bending and compression; used in the horizontal position, as a balcony beam for example, it becomes subject to combined bending and torsion. Reinforced concrete beams with appreciable horizontal projections are quite feasible if suitably designed for torsion. Perhaps the most striking example of torsion is the helical staircase, which has recently been revived in reinforced concrete after a long period of neglect.

CIRCULAR AND RECTANGULAR PLAIN CONCRETE BEAMS

Circular beams

Although circular concrete beams are extremely rare, the circular section is the only one for which a simple solution of the torsion problem can be obtained, and it is therefore briefly discussed.

Experiments show that when a circular section is twisted, plane sections remain plane, and the shear strain is therefore directly proportional to the distance from the neutral axis. Since, by definition, the shear stress equals the shear strain multiplied by the shear modulus of elasticity, the shear stress is also proportional to the distance from the neutral axis (Fig. 2). Consequently the torsional shear stress at some

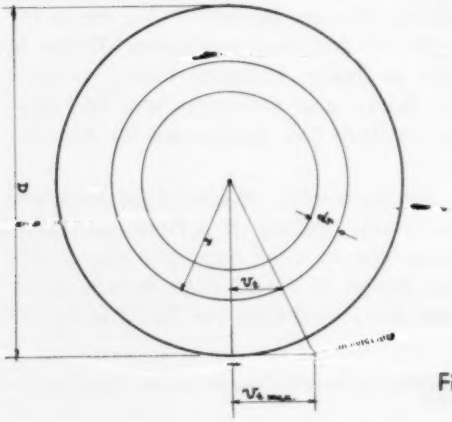

Fig. 2—Distribution of torsional shear stress in the circular section

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distance r from the neutral axis is

$$v_t = v_{t\,max} \frac{r}{\frac{1}{2}D}$$

This stress acts on an element of circumference $2\pi r$ and thickness dr, so that the area of the element is $2\pi r \times dr$. The torsional moment of the element about the neutral axis is the product of the stress in the element, the area on which it acts, and the distance r of this area from the neutral axis. The total twisting moment

$$M_t = \int_0^{\frac{1}{2}D} 2\pi r \cdot v_{t\,max} \cdot \frac{r}{\frac{1}{2}D} \cdot r\,dr = v_{t\,max}\,\pi \frac{D^3}{16} \quad \text{.....(1)}$$

where D is the diameter of the section, and $v_{t\,max}$ is the maximum torsional shear stress, which occurs at the periphery of the circle.

Rectangular beams

The problem of the torsion of noncircular sections is complicated by the warping of originally plane cross sections during twisting (Fig. 3), which produces axial as well as circumferential shear stress. The general theory of torsion is credited to St. Venant,* who also computed the numerical values of the principal torsion functions required for the design of several cross sections, including the rectangular section.

In a rectangular section the torsional shear stresses are nil at the corners and at the centroid of the rectangle. The maximum shear stresses occur on the periphery at the middle of the sides, the absolute maximum being at the middle of the longer sides of the rectangle

$$v_{t\,max} = \gamma x G_c \tau \quad \text{.....(2)}$$

where x = length of shorter side of rectangle
y = length of longer side of rectangle
γ = a function of the ratio y/x
G_c = shear modulus of elasticity of concrete
τ = angle of twist, measured in radians per unit length

The twisting moment

$$M_t = \beta x^2 y G_c \tau \quad \text{.....(3)}$$

where β is another function of the ratio y/x.

*M. de Saint-Venant's original paper (*Memoires presentes par Divers Savants a l'Academie des Sciences de l'Institut Imperial de France*, V. 14, Second Series, 1856, pp. 223-560) is now difficult to obtain. An account of St. Venant's theory can be found in most textbooks on the mathematical theory of elasticity, e.g., A. E. H. Love.[15] Some textbooks on the strength of materials give a summary of the theory, e.g., S. Timoshenko.[16]

Combining Eq. (2) and (3)

$$M_t = \alpha x^2 y v_{t\,max} \quad (4)$$

where $\alpha = \beta/\gamma$.

The numerical values of α, β, and γ are given in Table 1.

Since the horizontal and vertical shear stresses must be the same, this equation gives the magnitude of both.

The diagonal tensile stress is also numerically equal to the horizontal and vertical shear stresses. From Eq. (4) the maximum diagonal tensile stress in the concrete due to tension is

$$f_t = \frac{M_t}{\alpha x^2 y} \quad (5)$$

The value of α varies from 0.208, or approximately 1/5 for square sections, to 1/3 for thin rectangles. For $y/x = 2$, α is approximately 1/4.

The Australian code gives the formula

$$f_t = \frac{5\,M_t}{x^2 y} \quad (6)$$

for the maximum stress due to torsion in the absence of more precise calculations. This formula is correct for square sections, and somewhat conservative for rectangular sections of the type normally used in concrete construction.

Example 1

Calculate the maximum diagonal tensile stress in a section 12 in. wide by 24 in. deep due to a twisting moment of 50,000 in.-lb.

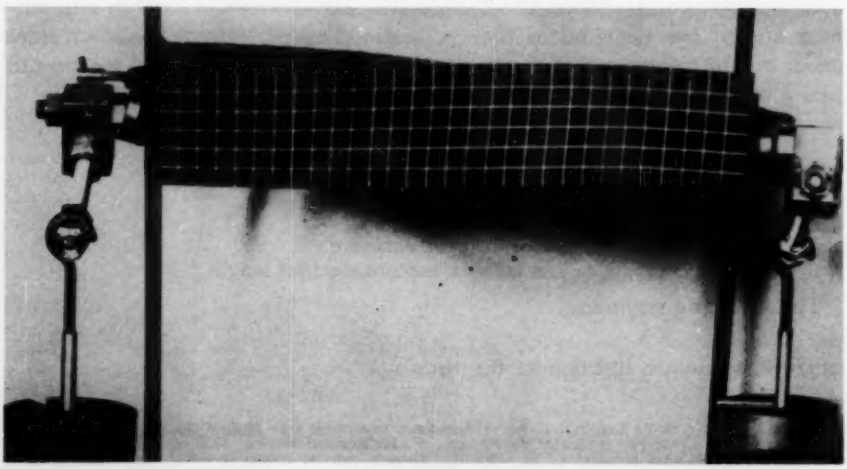

Fig. 3—Rubber model of a rectangular section subject to torsion

TABLE 1—ELASTIC TORSION THEORY CONSTANTS FOR RECTANGULAR SECTIONS, INCLUDING SECTIONS WITH SHEAR REINFORCEMENT

y/x or y_1/x_1	α	β	γ	λ
1.0	0.208	0.141	0.675	0.835
1.2	0.219	0.166	0.759	0.812
1.4	0.227	0.187	0.822	0.861
1.6	0.234	0.204	0.869	0.799
1.8	0.240	0.217	0.904	0.862
2.0	0.246	0.229	0.930	0.807
2.5	0.258	0.249	0.968	0.827
3.0	0.267	0.264	0.985	0.845
5.0	0.292	0.291	0.999	Not
10.0	0.312	0.312	1.000	computed
100.0	0.331	0.331	1.000	
∞	0.333	0.333	1.000	

Using the approximate formula given by the Australian code, the maximum diagonal tensile stress, from Eq. (6),

$$f_t = \frac{5 \times 50{,}000}{12^2 \times 24} = 72.3 \text{ psi}$$

A more accurate (and lower) answer is obtained from the exact theory. For $y/x = 24/12 = 2$ the value of α (from Table 1) is 0.246. From Eq. (5)

$$f_t = \frac{50{,}000}{0.246 \times 12^2 \times 24} = 58.8 \text{ psi}$$

Example 2

Determine the angle of twist of the beam of Example 1 over a length of 20 ft. The shear modulus of concrete may be taken as 900,000 psi

For $y/x = 2$, the value of $\beta = 0.229$, and from Eq. (3) the angle of rotation

$$\tau = \frac{500{,}000}{0.229 \times 12^3 \times 24 \times 900{,}000} = 5.85 \times 10^{-6} \text{ radians per in.}$$

Over a 20 ft length the angle of rotation is

$$5.85 \times 10^{-6} \times 20 \times 12 = 1.40 \times 10^{-3} \text{ radians} = 0.080 \text{ deg} = 0 \text{ deg } 5 \text{ min}$$

PLAIN CONCRETE BEAMS WITH FLANGES

T-, L-, and I-sections

Full numerical results have not so far been computed for T-, L-, and I-sections; however, an approximate assessment of the torsional moment is adequate for most practical problems.

When the torsional moment is small in relation to the other forces and moments acting on the member, it may be sufficient to compute the torsional resistance of the web only, and neglect the outstanding portions of the flange. The T- or L-section is thus reduced to a rectangular section (Fig. 4).

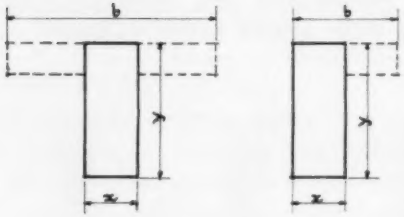

Fig. 4—Reduction of T- and L-sections to rectangular sections by neglecting the outstanding portions of the flange

A more accurate result is obtained from Bach's approximation,* which divides the section into its component rectangles (Fig. 5). The shear stresses attain maximum values at the middle of the longer sides of the component rectangles, the absolute maximum occurring in the thickest rectangle. The critical torsional shear stress occurs normally half-way down the web, because the web is usually thicker than the flange, and because there is usually some transverse shear stress due to bending which has a maximum at the neutral axis and must be added to the torsional shear stress. Since γ is approximately equal to 1.0 for values of $y/x > 2$, the maximum torsional shear stress in the web

$$v_{t\ max} = b' G_c \tau$$

where b' is the width of the web. From Table 1 it is evident that β tends toward 1/3 for high values of y/x, so that the torsional resistance moment of a section composed of a number of thin rectangles twisted through the same angle τ is

$$M_t = G_c \tau \Sigma \tfrac{1}{3} x^3 y$$

Combining these two equations

$$M_t = \frac{v_{t\ max}}{3\,b} \Sigma x^3 y \qquad (7)$$

The diagonal tensile stress is numerically equal to the horizontal and vertical shear stresses. Consequently the greatest diagonal tensile stress in T-, L-, and I-sections

$$f_t = \frac{3 M_t b'}{\Sigma x^3 y} \qquad (8)$$

where b' = width of web
$\Sigma x^3 y$ = sum of the terms $x^3 y$ of the component rectangles
x = smaller over-all dimension of rectangle
y = longer over-all dimension of rectangle

This is the equation given by the Australian code.

In the case of the T-section and the L-section shown in Fig. 5, the greatest diagonal tensile stress is given by

$$f_t = \frac{3 M_t b'}{b'^3 h\ (b - b')\ t^3} = \frac{3 M_t}{b'^3 h\ (\frac{b}{b'} - 1)\ t^3} \qquad (9)$$

*Bach, C., *Elastizitat und Festigkeit*, J. Springer, Berlin, 1911. An account is given in many textbooks on strength of materials, e.g., S. Timoshenko.[16]

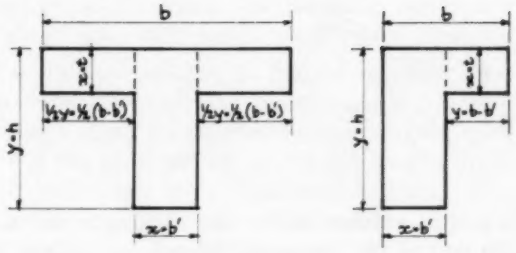

Fig. 5 — Approximate torsional resistance moment of T- and L-sections is obtained by dividing the sections into component rectangles

Example 3

A concrete T-beam has an over-all depth of 36 in. The web is 18 in. wide, and the flange is 8 ft wide by 6 in. thick. Determine the maximum diagonal tensile stress due to a twisting moment of 200,000 in.-lb, using the approximate formula of the Australian code.

Dividing the T-section into component rectangles as shown in Fig. 5, the maximum diagonal tensile stress, from Eq. (9)

$$f_t = \frac{3 \times 200{,}000}{18^2 \times 36 + \left(\frac{96}{18} - 1\right) \times 6^3} = \frac{600{,}000}{11{,}680 + 940} = 47.5 \text{ psi}$$

Eq. (8) and (9) introduce two errors which tend in opposite directions and therefore partially cancel one another. The value $\alpha = 1/3$ applies to a thin rectangle, whereas the web of a concrete T-beam (which contributes more to the torsional resistance moment than the flanges) usually has a y/x ratio between 2 and 3, corresponding to $\alpha = 1/4$. This error is on the unsafe side. The separation of the section into its component rectangles ignores the "junction effect," i.e., the additional torsional resistance due to the joints. This error is on the safe side.

Where the member is large and the twisting moment dominates the design, Eq. (8) and (9) are not sufficiently accurate. Exact values of the torsional strength according to the elastic theory can be obtained either by Southwell's relaxation method, which employs successive approximations, or by Prandtl's membrane analogy, which is based on the identity of the mathematical equations for torsion and for a membrane under pressure stretched over a hole of the same cross-sectional shape.*

Prandtl's membrane analogy gives a clear visual impression of the error resulting from the neglect of the "junction effect." Fig. 6 shows a membrane stretched over the end part of a rectangle. There is evidently

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*Southwell, R. V., *Relaxation Methods in Theoretical Physics*, Oxford University Press, 1946. Prandtl, L., *Physikalische Zeitschrift*, V. 4, 1903, pp. 758-759. The result is obtained by experiment. A hole of the same shape as the cross section of the section is cut in one side of a box, and a membrane is stretched over it, the best results being obtained with a soap film. The air pressure in the box is then increased (e.g., with a bicycle pump), and the membrane rises under the pressure. The torsional resistance moment of the cross-sectional shape is proportional to the volume contained under the raised membrane, and the shear stress is proportional to the greatest slope of the membrane.

An account of the application of both methods to steel I-sections is given by W. F. Cassie and W. B. Dobie,[17] who also list a number of approximate formulas, which are more complex, but more accurate, than the Bach approximation.

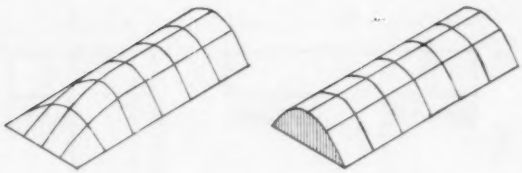

Fig. 6—The junction effect. Left — Membrane stretched over the end part of a rectangle. Right — Membrane stretched over the central portion of a long rectangle

a loss of volume under the rectangle due to the restraint exercised by the end of the rectangle. If two rectangles were joined end to end, the volume would be increased as shown on the right in Fig. 6. A similar result would be obtained from joining two rectangles into a T- or L-beam.

While the elastic theory gives the correct torsional resistance moment, it does not give the correct value for the maximum shear stress. A re-entrant corner results in high stresses over a small area near the corner even at low loads. Since the elastic theory is based on the assumption that stress is proportional to strain, the theory leads to the conclusion that at moderate loads the maximum torsional shear stress is far beyond the ultimate strength of the material. For a T-section with a perfectly sharp re-entrant corner the elastic theory predicts infinitely high torsional shear stresses at the corner at the lowest loads; the shear stress at a rounded re-entrant corner depends on the radius of curvature of the corner. In practice these high stresses are not realized because materials cease to behave elastically at high stresses. In the case of a steel section complete redistribution of stresses at the corner occurs due to plastic yielding of the material at a constant stress. The yielding is confined to a small area of the section, and does not affect its general resistance to torsion. In the case of concrete, which does not yield plastically, there is, nevertheless, a substantial redistribution of stress.[6,8,16]

An accurate formula for the torsional strength must therefore allow for the relatively low values of the ratio y/x for the web of concrete sections, for the effect of the junction between the component rectangles, and for the redistribution of stress at the re-entrant corners. Although a number of equations have been derived which are more accurate than the Bach formula,[10,17] they are all complicated and time-consuming in their application.

An analysis of experiments on the torsional strength of reinforced concrete has demonstrated[10,18] that near the ultimate load redistribution of stress occurs not only at the re-entrant corners, but over the entire section. The experimental data are reasonably consistent with the assumption of uniform shear stress over the whole section. It is therefore possible to apply the plastic theory to concrete T- and L-beams. This results in much simpler formulas and some economy of material. The use of equations based on ultimate strength for the torsional resistance of T- and L-beams under working loads does not in-

troduce any new fundamental principles since the design of concentrically loaded columns under working loads is based on ultimate strength.

Since it is assumed that the shear stress due to plastic redistribution of stress is uniform over the entire section, the plastic stress function is a surface of constant maximum slope which may be constructed over the edge of the cross section.[19] If the cross section is cut out from a piece of cardboard or sheet metal, suspended in a horizontal position, and covered with sand until it spills over the edges, the natural slope of the sand gives a picture of the plastic torsion function. Alternatively it may be visualized as a pitched roof of constant slope erected over a building which is T-shaped and L-shaped in plan. The torsional resistance moment of the cross-sectional shape is proportional to the volume contained in the sand heap or under the roof, and the shear stress is proportional to the constant slope of the sand heap or the roof.

The volume can be readily worked out for any shape composed of rectangles. The maximum diagonal tensile stress for T-sections[10]

$$f_t = \frac{M_t}{\frac{1}{2}\,t^2\,(b - \frac{1}{3}\,t) + \frac{1}{2}\,b'^2\left[h - t - \frac{1}{3}\,b' + 0.86\,\frac{b'^2 t}{b'^2 + t^2}\right]} \qquad (10)$$

with the notation shown in Fig. 5.

Similarly for L-sections

$$f_t = \frac{M_t}{\frac{1}{2}\,t^2\,(b - \frac{1}{3}t) + \frac{1}{2}\,b'^2\left[h - t - \frac{1}{3}\,b' + 0.79\,\frac{b'^2 t}{b'^2 + t^2}\right]} \qquad (11)$$

Example 4

Determine the maximum diagonal tensile stress for the T-beam of Example 3, using the ultimate strength theory.

Eq. (10) takes account of the junction effect and of the correct ratio of overall depth to web thickness.*

$$f_t = \frac{200{,}000}{\frac{1}{2} \times 6^2(96 - \frac{6}{3}) + \frac{1}{2} \times 18^2\left[36 - 6 - \frac{18}{3} + 0.86\,\frac{18^2 \times 6}{18^2 + 6^2}\right]}$$

$$f_t = \frac{200{,}000}{18 \times 94 + 162\,(36 - 6 - 6 + 4.6)} = \frac{200{,}000}{1690 + 4630} = 31.6 \text{ psi}$$

Box section

When the twisting moment is large, a box section gives the most economical section. A thin tube evidently is the most efficient section for

*Eq. (9) applies strictly only to very thin component rectangles.

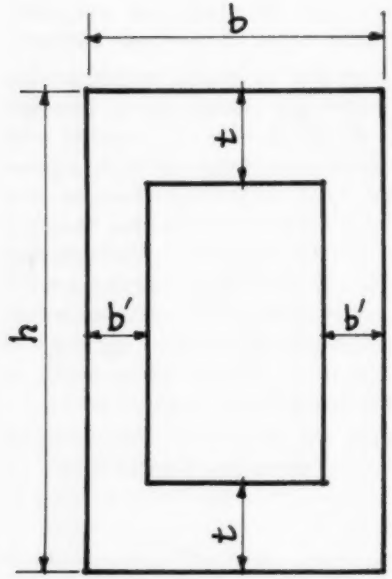

Fig. 7—Dimensions of rectangular box section

resistance to torsion since it concentrates the entire cross-sectional area in the most highly stressed region, and the rectangular box is the practical concrete section most closely approximating it. The torsional strength of the tube is far greater than the sum of the strengths of its component parts, and Eq. (8) is therefore seriously in error.

The maximum shear stress in a thin tube is given by[20]

$$v_{max} = \frac{M_t}{2At}$$

where M_t = twisting moment
t = thickness of the tube
A = mean of the areas enclosed by the inner and outer boundaries of the tube

As before, the diagonal tensile stress is numerically equal to the shear stress.

Consequently the maximum diagonal tensile stress at the middle of the larger side of the rectangular box-section shown in Fig. 7* is,

$$f_t = \frac{M_t}{2b'(h-t)(b-b')} \qquad (12)$$

Example 5

A rectangular box section has an over-all depth of 48 in. and an over-all width of 36 in. The concrete walls are 6 in. thick on both the horizontal and the vertical parts of the box. Determine the maximum diagonal tensile stress due to a twisting moment of 500,000 in.-lb.

The dimensions of the section are $b = 36$ in.; $h = 48$ in.; $b' = 6$ in.; and $t = 6$ in.

From Eq. (12) the maximum diagonal tensile stress at the middle of the longer side of the box section is

$$f_t = \frac{500{,}000}{2 \times 6 \times (48-6)(36-6)} = 33.0 \text{ psi}$$

*The "membrane" and the "saddle roof" over a *thin* box-section cut-out enclose virtually the same volume, so that almost identical results are obtained from the elastic and the elastic theory.

Bach's approximate formula is seriously in error for the box section. From Eq. (8)

$$f_t = \frac{3M_t b'}{2b'^2 h + 2\,(b - 2b')\,t^2}$$

$$f_t = \frac{3 \times 500{,}000 \times 6}{2 \times 6^2 \times 48 + 2 \times 24 \times 6^2} = 289 \text{ psi}$$

TORSIONAL SHEAR REINFORCEMENT

If concrete is twisted, it fails along a spiral line at 45 deg to the axis when the maximum diagonal tensile stress exceeds the tensile strength of the material (Fig. 8). Failure can be delayed, but not entirely prevented, by shear reinforcement crossing the lines of the potential cracks.

The most effective reinforcement consists of a series of 45-deg spirals. Spiral reinforcement has, however, two disadvantages: rectangular spirals are not easily made (Fig. 9), and two series of spirals, right hand and left hand, are required in beams which are subject to a reversal of twisting moment. In practice the shear reinforcement is most conveniently provided in the form of horizontal bars and of vertical hoops, as in the case of transverse shear reinforcement. The hoops resist the vertical component of the diagonal tension, and the longitudinal bars resist the horizontal component (Fig. 10).

Torsion differs from transverse shear in one important respect: diagonal tensile stresses exist on all four faces of a rectangular section subject to torsion, whereas they only extend over the two vertical faces in a section subject to transverse shear. Consequently U-stirrups and bent-up bars are unsuitable for torsional shear reinforcement, and closed hoops, properly anchored at the ends, must be used. Inclined hoops running in the direction of the diagonal tension on one face, would run in the wrong direction on the opposite face; hoops must therefore be vertical. Similarly the longitudinal reinforcement must be suitably dis-

Fig. 8—Failure of plain concrete in pure torsion

tributed over all four faces, and the Australian code requires at least one bar to be placed in the corner of each hoop.

Although circular beams are rare, and spiral reinforcement is not common, let us consider a circular beam reinforced with N continuous 45-deg spirals at a pitch s (Fig. 11), since it is the simplest case of torsional shear reinforcement. If the diameter of the spiral cage is x_1, measured to the centers of the spiral bars, each spiral makes a complete turn over a length

$$l = \pi x_1$$

of the beam, and the number of continuous spiral bars

$$N = l/s = \pi x_1/s$$

Then tensile force in each spiral bar is

$$T = f_s a_s$$

where f_s is the (uniform) stress in the spiral bar and a_s is its cross-sectional area.

When a twisting moment is applied to the beam, it sets up diagonal tensile forces T in the spirals and diagonal compressive forces C in the concrete at right angles, as shown in Fig. 11. These forces form a rectangular lattice which resists the torsion. The axial components of these forces are equal, since there are no other forces with axial components.

$$T \cos 45 \text{ deg} = C \sin 45 \text{ deg}$$

Since $\cos 45 \text{ deg} = \sin 45 \text{ deg} = 1/\sqrt{2}$,

$$T = C$$

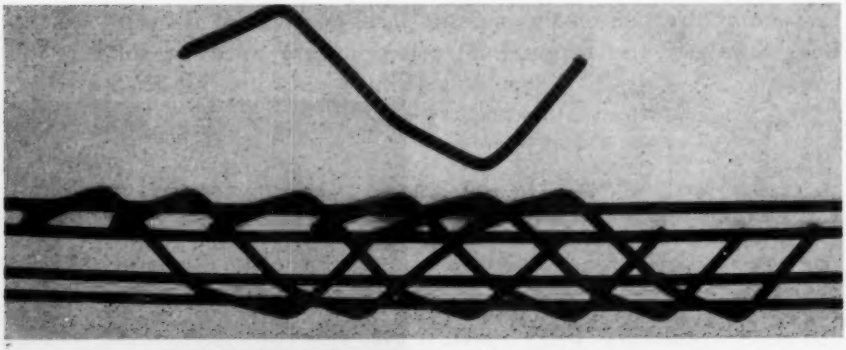

Fig. 9—Model of rectangular spiral reinforcing cage

Spiral reinforcement is easily wound for circular beams. For rectangular beams, however, three-dimensional stirrups are required, and the process of manufacture is complicated, even when individual spiral stirrups are joined by welding as shown in this illustration

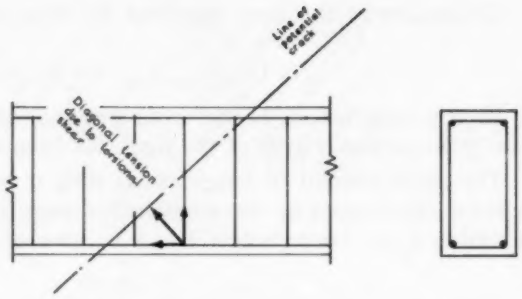

Fig. 10—Vertical hoops and longitudinal bars used as torsional shear reinforcement. The longitudinal bars resist the horizontal component and the hoops the vertical component of the diagonal tension

The moment of the circumferential components of the forces T and C about the axis of the circular beam (radius $\frac{1}{2}\ x_1$) equals the additional twisting moment M_{t_s} due to the insertion of the continuous spirals.

$$M_{t_s} = N(T \sin 45 \text{ deg} + C \cos 45 \text{ deg}) \cdot \tfrac{1}{2}x_1$$

$$M_{t_s} = N\left(\frac{T}{2} + \frac{C}{\sqrt{2}}\right) \cdot \tfrac{1}{2}x_1 = N\frac{T}{\sqrt{2}}x_1$$

since the diagonal forces T and C are equal.

Substituting for N and T

$$M_{t_s} = \frac{\pi x_1}{s} \cdot \frac{f_s a_s}{\sqrt{2}} \cdot x_1 = \frac{f_s a_s \pi x_1^2}{\sqrt{2}\ s}$$

The minimum area of spiral reinforcement is obtained when it is stressed to its maximum permissible stress.

$$a_s = \frac{\sqrt{2}\, M_{t_s}\, s}{f_s\, \pi x_1^2} \qquad (13)$$

In this equation a_s represents the cross-sectional area of *one* bar or wire.

Let us consider next the circular section reinforced with circular hoops and longitudinal bars. The diagonal tensile forces due to torsion are then resisted by the components of the tensile forces in the hoops and in the longitudinal steel in the direction of the diagonal tension, while the diagonal compressive forces are resisted by the concrete, as as in the case of spiral reinforcement.

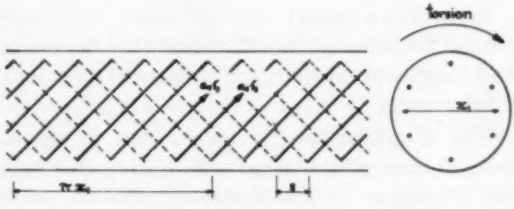

Fig. 11 — Circular section with spiral reinforcement
When a twisting moment is applied to the beam, it sets up diagonal tensile forces T in the spirals, and diagonal compressive forces C in the concrete at right angles. These forces form a rectangular lattice which resists the torsion a_s = cross-sectional area of one bar or wire in the spiral cage

Consequently the area required for the vertical hoops

$$a_{sv} = \frac{a_s}{\sin 45 \text{ deg}} = \sqrt{2}a_s = \frac{2M_{ts}s}{f_s \pi x_1^2} \quad \text{.....(14)}$$

In this equation a_{sv} is the cross-sectional area of *one* bar or wire, i.e., the cross-sectional area of the material from which the hoops are made.

The same amount of longitudinal steel is required to resist the longitudinal component of the longitudinal tensile forces. A length of beam s contains one hoop, which has a volume of

$$a_{sv}\pi x_1$$

The same volume of longitudinal steel is needed in the same length of beam

$$A_{sl}s = A_{sv}\pi x_1$$

In this equation A_{sl} is the cross-sectional area of *all* the longitudinal bars, which is therefore

$$A_{sl} = a_{sv}\frac{\pi x_1}{s} \quad \text{.....(15)}$$

The problem of the rectangular section is much more complicated, partly because of the complexities of the torsion theory for noncircular sections, and partly because the stress in the shear reinforcement is not constant, as in the circular section. Provided that bond with the concrete is maintained, it varies with the stress in the surrounding concrete. The maximum steel stress then occurs in the middle of the longer sides; another smaller maximum steel stress occurs at the middle of the shorter sides of the rectangle, and the steel stress is negligibly small at the corners. A solution has been obtained by equating the strain energy stored in the spirals and in the concrete in diagonal compression to the work done by the twisting moment in rotating the beam.[21] This gives the following area for the spiral reinforcement for rectangular beams (Fig. 12)

$$a_s = \frac{\sqrt{2}\,M_{ts}\,s}{4\lambda\, f_s\, x_1\, y_1} \quad \text{.....(16)}$$

where a_s is the cross-sectional area of *one* bar or wire in the spiral cage, x_1 and y_1 are the smaller and larger dimensions, respectively, of the spiral cage, measured from center to center, and λ is a function of the ratio y_1/x_1 (Table 1).

In practice spiral reinforcement is hardly ever used in rectangular beams because of the difficulty of manufacture. Vertical closed hoops and longitudinal bars are a more convenient form of torsional shear reinforcement.

The diagonal tensile forces due to torsion are then resisted by the components of the tensile forces in the hoops and longitudinal steel in the direction of the diagonal tension, while the diagonal compressive

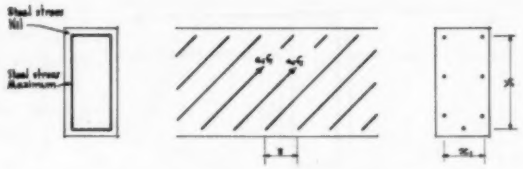

Fig. 12 — Rectangular section with spiral reinforcement. a_s = cross-sectional area of *one* bar or wire in the spiral cage

forces are resisted by the concrete. Consequently the area required for the vertical hoops is

$$a_{sv} = \frac{a_s}{\sin 45 \text{ deg}} = \sqrt{2}\, a_s = \frac{M_t\, s}{2\lambda\, f_s\, x_1\, y_1}$$

Since in rectangular beams the same hoops may be used for torsional shear reinforcement and for transverse shear reinforcement, it is convenient to use the same notation for both. In the equations for transverse shear reinforcement A_{sv} denotes the area of *all* the shear reinforcement at a given section, i.e., the area of two bars in the base of hoops, so that

$$A_{sv} = 2a_{sv} = \frac{M_t\, s}{\lambda\, f_s\, x_1\, y_1} \qquad (17)$$

The value of λ does not vary greatly with the ratio x_1/y_1 (Table 1) and $\lambda = 0.8$ is sufficiently accurate for all practical purposes.

$$A_{sv} = \frac{M_t\, s}{0.8\, f_s\, x_1\, y_1} \qquad (18)$$

where A_{sv} = cross-sectional area of *two* legs of the closed hoop
s = pitch of the hoops
x_1 and y_1 = the smaller and larger dimensions of the hoops, respectively, measured center to center
f_s = maximum permissible steel stress
M_t = additional twisting moment due to the insertion of the hoops and a corresponding amount of longitudinal steel

The same amount of longitudinal steel is required to resist the horizontal component of the diagonal tension.
A length of beam s contains one hoop, which has a volume

$$A_{sv}\,(x_1 + y_1)$$

An equal volume of longitudinal steel is needed in the same length of beam.

$$A_{sl}\, s = A_{sv}\,(x_1 + y_1)$$

In this equation A_{sl} denotes the cross-sectional area of all the longitudinal bars, which is therefore

$$A_{sl} = A_{sv}\,\frac{x_1 + y_1}{s} \qquad (19)$$

Eq. (18) and (19) are the formulas given by the Australian code for the design of torsional shear reinforcement. The code requires

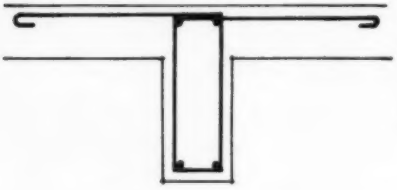

Fig. 13 — When hoops are extended well into the flange, it is reasonable to allow the additional twisting moment due to the rectangular hoops and longitudinal bars to be added to that of the plain concrete T- or L-beam

at least one longitudinal bar to be placed in each corner of the hoops. The hoops must either be adequately secured by bond and anchorage, or by welding to the main reinforcement. In practice it is not easy to achieve adequate anchorage in rectangular sections by bond and hooks, since torsional shear, unlike transverse shear, occurs on all four faces of a rectangular section. Where twisting moments are considerable, welding of the hoops of the longitudinal steel is therefore necessary.

Hoops and longitudinal bars can be placed in the webs of T- and L-beams, and the flange allows ample room for the anchorage. When the hoops are extended well into the flange as shown in Fig. 13, it is reasonable to allow the additional twisting moment due to the rectangular hoops and longitudinal bars to be added to that of the plain concrete T-beam.

MAXIMUM PERMISSIBLE STRESSES IN TORSION, COMBINED TORSION AND SHEAR, AND COMBINED TORSION AND BENDING

Since torsion produces shear stresses, and torsion reinforcement is designed to resist the diagonal tension due to shear, the maximum permissible stresses for torsion should be similar to those for transverse shear. The Australian code limits the maximum permissible concrete stress in diagonal tension due to torsion to

$$f_t = 0.02\, f_c' + 20 \text{ psi} \quad \leq 90 \text{ psi}$$

Since concrete weaker than $f_c' = 1800$ psi is not permitted by the Australian code, the permissible diagonal tensile stress can vary only within the range of 56 and 90 psi.

When this stress is exceeded, torsion reinforcement must be provided to take the excess diagonal tension, but irrespective of the amount of torsion reinforcement, the diagonal tensile stress must not exceed

$$f_t = 0.08\, f_c' + 80 \text{ psi} \quad \leq 360 \text{ psi}$$

These limiting stresses are the same as for transverse shear. The Australian code does not, however, specify a limiting stress above which the entire diagonal tension must be resisted by reinforcement. In the case of transverse shear the entire concrete below the neutral axis is stressed to the maximum diagonal tensile stress, so that the cracks

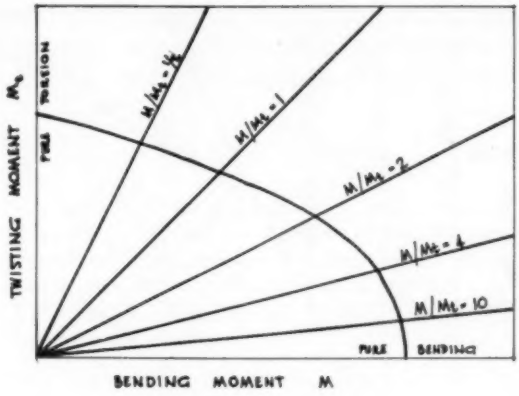

Fig. 14 — Relation between ultimate bending moment and ultimate twisting moment for plain concrete

extend right across the section below the neutral axis. In circular and rectangular sections subject to torsion, however, the maximum diagonal tensile stresses occur only in a surface layer, and the stresses in the core of the section are small. The concrete may therefore be assumed to contribute to the torsional resistance moment of the section even when the stresses at the surface of the section are comparatively high.

Torsion in reinforced concrete construction is commonly a secondary effect of transverse loading, and sections subject to torsion are therefore frequently also subject to transverse shear and to bending.

Transverse shear forces set up diagonal tensile stresses which act in the same direction on the two vertical faces of a beam, while diagonal tensile stresses due to torsion act in opposite directions on opposite faces, like a spiral. In consequence, the diagonal tensile stresses due to transverse shear and due to torsion partly cancel one another on one face, and reinforce one another on the other, which is the critical face for the design of the beam.

The Australian code therefore provides that the combined stress due to transverse shear and torsion must not exceed

$$f_s + f_t = 0.08\, f_c' + 80 \text{ psi} \quad \leq 360 \text{ psi}$$

irrespective of the amount of transverse and torsional shear reinforcement.

Bending moments do not reduce the capacity of reinforced concrete sections to resist torsion. In this respect reinforced concrete differs from steel and from plain concrete (Fig. 14). In fact, a moderate bending moment increases the resistance to torsion[10,12] (Fig. 15).

The reason for this interaction between bending moments and twisting moments in reinforced concrete is not entirely clear. It seems to be due to the essential difference in the failure of reinforced concrete in torsion and in bending.[22-24] Failure in torsion depends on the diagonal tensile stress. The formation of a tension crack does not, however, affect the

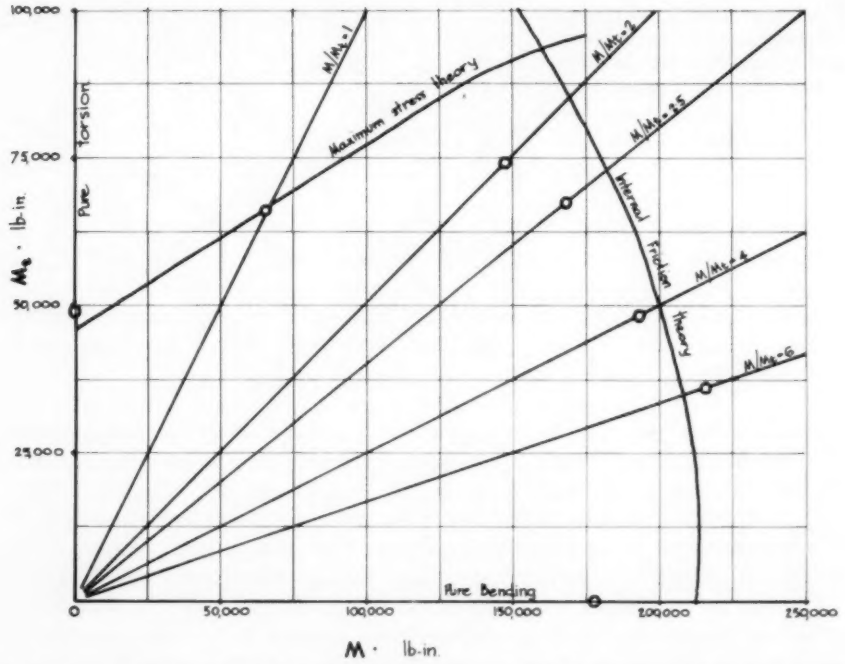

Fig. 15—Relation between bending moment and twisting moment for reinforced concrete at the elastic limit.[12] The symbol O represents experimental results

resistance to bending; in fact, tension cracks are always present in reinforced concrete beams under the action of the working loads. Failure in bending can occur only as a result of a compression failure in the concrete, either directly or as a result of yielding of the tension reinforcement.

Two criteria of failure are therefore used in Fig. 15: the maximum stress theory for the primary tension failure due to torsion, and the internal friction theory for the primary compression failure due to bending.[25] Some of the test specimens from the series of experiments plotted in Fig. 15 are shown in Fig. 16. These show the existence of two different types of cracks: vertical cracks below the neutral axis due to bending, and diagonal tension cracks due to tension.

The Australian concrete code does not call for any reduction of the maximum permissible stresses for torsion if bending is present, or vice versa.* This provision is shown by the dotted lines in Fig. 17, the full

*It should be noted that steel codes normally require a reduction in the maximum permissible stresses for both bending and torsion when a section is subject to combined bending and torsion.

lines indicating the corresponding experimental relationship. It evidently provides an additional margin of safety for sections with relatively large twisting moments which are likely to need heavy torsion reinforcement.

Example 6

A rectangular reinforced concrete section, 12 in. wide by 24 in. deep, is subject to a twisting moment of 100,000 in.-lb. Calculate the amount of torsion reinforcement required, if the maximum permissible diagonal tensile stress for the concrete is 64 psi and the maximum permissible steel stress is 18,000 psi.

From Table 1, the coefficient $\alpha = 0.246$ for $y/x = 24/12 = 2$, and from Eq. (5) the twisting moment resisted by the concrete alone is

$$M_{tc} = f_t \alpha x^2 y = 64 \times 0.246 \times 12^2 \times 24 = 54{,}400 \text{ in.-lb}$$

The additional twisting moment for which reinforcement must be provided is therefore

$$M_{ts} = 100{,}000 - 54{,}400 = 45{,}600 \text{ in.-lb}$$

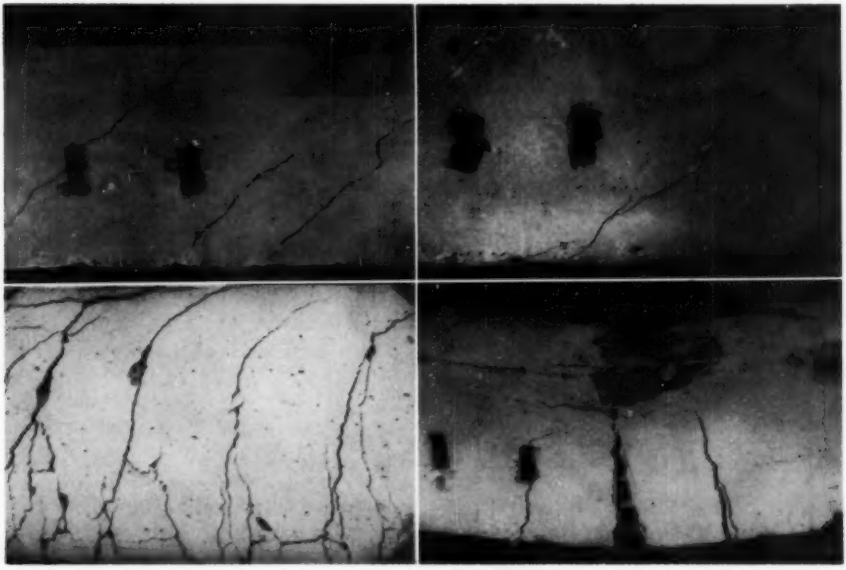

Fig. 16—Primary torsion and primary bending failure of reinforced concrete beams in combined bending and torsion

Primary torsion failures

(Left, above) — Pure torsion. (Right, above) $\dfrac{\text{Bending moment}}{\text{Twisting moment}} = 1$

Primary bending failures

(Left, below) — $\dfrac{\text{Bending moment}}{\text{Twisting moment}} = 2\frac{1}{2}$. (Right, below) — Pure bending

For the purpose of estimating the width and depth of the hoops, we will assume a ½-in. diameter, so that

$$x_1 = \underset{\left(\begin{smallmatrix}\text{over-all}\\ \text{width}\end{smallmatrix}\right)}{12} - \underset{\left(2 \times \text{cover}\right)}{2 \times 1} - \underset{\left(\begin{smallmatrix}\text{hoop}\\ \text{diameter}\end{smallmatrix}\right)}{½} = 9.5 \text{ in.}$$

$$y_1 = 24 - 2 \times 1 - ½ = 21.5 \text{ in.}$$

From Eq. (18) the cross-sectional area of the vertical hoops is

$$A_{sv} = \frac{45{,}600 \times s}{0.8 \times 18{,}000 \times 9.5 \times 21.5} = 0.0155\ s$$

Therefore ¼-in. diameter hoops are evidently sufficient. The area of two legs of the hoops, $A_{sv} = 0.098$, and the pitch $s = 0.098/0.0155 = 6.32$ in. — say ¼-in. diameter hoops at 6 in. on centers.

The area of longitudinal steel required, from Eq. (19)

$$A_{sl} = (x_1 + y_1)$$

$$A_{sl} = 0.0155\ (9.5 + 22.5) = 0.481 \text{ sq in.}$$

Say four ½-in. diameter bars, i.e., one bar in each corner of the hoops (0.785 sq in.)

Example 7

A reinforced concrete T-beam has an over-all depth of 36 in. The web is 18 in. wide and the flange is 8 ft wide by 6 in. thick. Calculate the amount of torsion reinforcement required, if the beam is subject to a twisting moment of 400,000 in.-lb. The permissible stresses are the same as in Example 6.

The twisting moment resisted by the concrete alone, from Eq. (9) is

$$M_{tc} = 64 \times ⅓\ [18^2 \times 36 + (^{96}⁄_{18} - 1) \times 6^3] = 270{,}000 \text{ in.-lb}$$

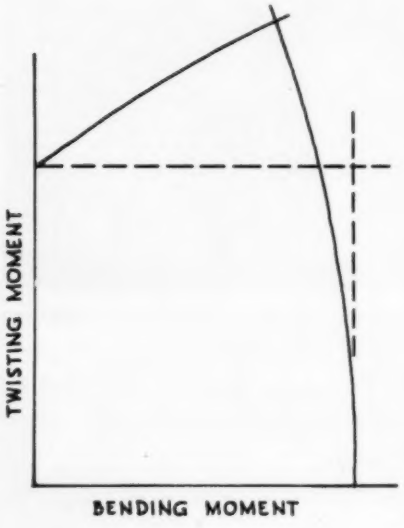

Fig. 17—Actual variation of bending and twisting moments, and variation for design in accordance with the Australian code. Solid line shows actual variation, dotted line shows variation for design

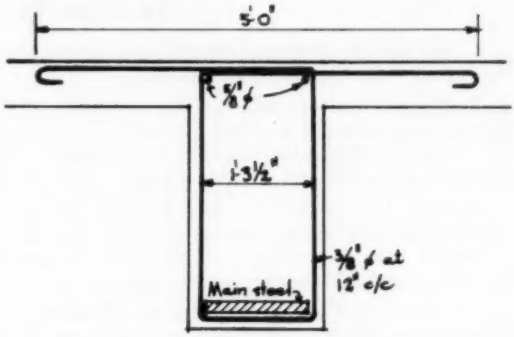

Fig. 18—Layout of the torsion reinforcement in the section of Example 7

The additional twisting moment for which reinforcement must be provided is therefore

$$M_{t_2} = 400{,}000 - 270{,}000 = 130{,}000 \text{ in.-lb}$$

The reinforcement is placed in the web which is 18 in. wide and 36 in. deep. Assuming ½-in. diameter bars for the purpose of estimating the width and depth of the hoops

$$x_1 = 18 - 2 \times 1 - \tfrac{1}{2} = 15.5 \text{ in.}$$

$$y_1 = 36 - 2 - \tfrac{1}{2} = 33.5 \text{ in.}$$

From Eq. (18) the cross-sectional area of the vertical hoops is

$$A_v = \frac{130{,}000 \times s}{0.8 \times 18{,}000 \times 15.5 \times 33.5} = 0.0174\ s$$

Using ⅜-in. diameter hoops, the area of two legs, $A_v = 0.220$, and the pitch $s = 0.220/0.0174 = 12.6$ in. — say ⅜-in. diameter hoops at 12 in. on centers.

The area of longitudinal steel required, from Eq. (19)

$$A_l = \frac{A_v}{s}(x_1 + y_1)$$

$$A_l = 0.0174\ (15.5 + 33.5) = 0.852 \text{ sq in.}$$

Say four ⅝-in. diameter bars

In practice there is likely to be a large area of longitudinal reinforcement on the tension face, so that the additional two 5/8-in. diameter bars are unnecessary. They must, however, be provided on the compression face.

It is important to provide proper anchorage of the hoops in the flange. The flange is 8 ft wide and the hoops will be extended over the central 5 ft and hooked at the ends. This ensures an ample length of anchorage for the hoops. The arrangement of the reinforcement is shown in Fig. 18.

REFERENCES

1. Mörsch, E., *Concrete-Steel Construction*, Engineering News Publishing Co., New York, 1910, pp. 39-40 and 53-55.

2. Bach, C., and Graf, O., "Experiments on the Resistance of Concrete and Reinforced Concrete Against Torsion" Versuche über die Widerstandsfähighkeit von Beton und Eisenbeton gegen Verdrehung), *Heft* No. 16, Deutscher Ausschuss für Eisenbeton (Wilhelm Ernst, Berlin), 1912, 78 pp.

3. Graf, O., and Mörsch, E., "Torsion Tests Conducted to Establish the Shear Strength of Reinforced Concrete" (Verdrehungsversuche zur Klärung der Schubfestigkeit von Eisenbeton), *Reports on Research in the Field of Engineering* No. 258, Verein Deutscher Ingenieure (Julius Springer, Berlin), 1922, 52 pp.

4. Young, C. R.; Sagar, W. L.; and Hughes, C. A., "The Torsional Strength of Rectangular Sections of Concrete, Plain and Reinforced," *Engineering Research Bulletin* No. 3, University of Toronto, 1922, pp. 145-169.

5. Miyamato, T., "Torsional Strength of Reinforced Concrete," *Concrete and Constructional Engineering* (London), V. 22, 1957, pp. 637-647.

6. Turner, L., and Davies, V. C., "Plain and Reinforced Concrete in Torsion, with Particular Reference to Reinforced Concrete Beams," *Selected Engineering Paper* No. 165, Institution of Civil Engineers (London), 1934, 31 pp.

7. Andersen, P., "Experiments with Concrete in Torsion," *Transactions*, ASCE, V. 100, 1935, pp. 945-983.

8. Marshall, W. T., and Tembe, N. R., "Experiments on Plain and Reinforced Concrete in Torsion," *Structural Engineer* (London), V. 19, 1949, pp. 171-191.

9. Cowan, H. J., "Tests of the Torsional Strength and Deformation of Rectangular Reinforced Concrete Beams," *Concrete and Constructional Engineering* (London), V. 46, 1951, pp. 51-59.

10. Nylander, H., "Torsion and Torsional Restraint of Concrete Structures" (Vridning och vridningsinspänning vid betongkonstruktioner), *Meddelanden* No. 3, Statens Kommitté för Byggnadsforskning (Stockholm), 1945, 138 pp.

11. Fisher, D., "The Strength of Concrete in Combined Bending and Torsion," Thesis for the degree of Ph.D., University of London, 1950.

12. Cowan, H. J., and Armstrong, S., "Experiments on the Strength of Reinforced and Prestressed Concrete Beams and of Concrete-Encased Steel Joists in Combined Bending and Torsion," *Magazine of Concrete Research* (London), V. 7, 1955, pp. 3-20.

13. Ernst, G. C., "Ultimate Torsional Properties of Rectangular Reinforced Concrete Beams," ACI JOURNAL, V. 29, No. 4, Oct. 1951 (*Proceedings* V. 54), pp. 341-356.

14. Measor, E. O., and New, D.H., "The Design and Construction of the Royal Festival Hall, South Bank," *Journal*, Institution of Civil Engineers (London), V. 36, 1951, pp. 241-318.

15. Love, A. E. H., *Mathematical Theory of Elasticity*, Cambridge University Press, Cambridge, 4th Edition, 1927, pp. 310–328.

16. Timoshenko, S., *Strength of Materials*, MacMillan, London, V. 1, 1938, pp. 67-91.

17. Cassie, W. F., and Dobie, W. B., "The Torsional Stiffness of Structural Sections," *Structural Engineer* (London), V. 26, 1948, pp. 154-182.

18. Marshall, W. T., "The Torsional Resistance of Plastic Materials with Special Reference to Concrete," *Concrete and Constructional Engineering* (London), V. 34, 1944, pp. 83-88.

19. Nadai, A., *Theory of Flow and Fracture of Solids*, McGraw-Hill Book Co., New York, 1950, pp. 490-511.

20. Case, J., *The Strength of Materials*, Edward Arnold, London, 1938, pp. 476-488.

21. Cowan, H. J., "Torsion of a Rectangular Elastic Isotropic Beam Reinforced with Rectangular Helices of Another Material," *Applied Scientific Research* (The Hague), V. 3A, 1953, pp. 344-348.

22. Cowan, H. J., "The Strength of Plain, Reinforced, and Prestressed Concrete Under the Action of Combined Stresses, with Particular Reference to the Combined Bending and Torsion of Rectangular Sections," *Magazine of Concrete Research* (London), V. 5, 1953, pp. 75-86.

23. Cowan, H. J., and Armstrong, S., "Reinforced Concrete in Combined Bending and Torsion," *Preliminary Publication*, Fourth Congress, International Association for Bridge and Structural Engineering, Cambridge, 1952, pp. 861-870.

24. Cowan, H. J., "Torsion in Reinforced and Prestressed Concrete Beams," *Journal*, Institution of Engineers (Australia), V. 28, 1956, pp. 235-240.

25. Cowan, H. J., "Strength of Reinforced Concrete Under the Action of Combined Stresses, and the Representation of the Criterion of Failure by a Space Model," *Nature* (London), V. 169, 1952, p. 663.

26. Buckton, E. J., and Cuerel, J., "The New Waterloo Bridge," *Journal*, Institution of Civil Engineers (London), V. 20, 1943, pp. 145-203.

APPENDIX — BIBLIOGRAPHY ON TORSIONAL STRENGTH

1. Mörsch, E., *Der Eisenbetonbau*, Berlin, 1903 (in German).

Includes a report of tests on three circular plain concrete specimens, 10.24 in. in diameter. The maximum shear stress was 243 psi.

2. Mesnager, Paris, 1903 (in French).

Report on two circular specimens, 4.21-in. diameter, longitudinally reinforced.

3. Föppl, A., Munich, 1911. (in German).

Report on 15 circular specimens, 3.93-in. diameter, plain and longitudinally reinforced concrete.

4. Bach, C., and Graf, O., "Experiments on the Torsional Strength of Concrete and Reinforced Concrete," *Heft* No. 16, Deutscher Ausschuss für Eisenbeton, Berlin, 1912. (in German).

Carried out at the laboratories of Stuttgart College of Technology, this was the first thorough investigation.

Plain concrete -- The experiments were carried out on specimens of circular, hollow circular, square, and rectangular cross section, with subsidiary direct tension and compression tests. The concrete used was mixed in the proportions 1:2:3, with a water-cement ratio of approximately 0.54. Beams were tested 45 days after casting.

The maximum shear stress varied from 243 to 462 psi. The ratio of torsional shear strength to compressive strength varied from 0.07 to 0.13. The ratio of torsional shear strength to tensile strength varied from 0.92 to 1.75; the highest value being in each case for the rectangular, and the lowest for the hollow circular section.

Torsional computations were based on Bach's approximate theory. Average value for modulus of rigidity at low loads was 1,920,000 psi. Poisson's ratio varied from 0.29 (at low loads) to 0.14 (at 170 psi).

Reinforced concrete — 11.81 in. square, and 8.26 x 16.52 in. rectangular reinforced specimens were tested.

Type of specimen tested	Ultimate torque
Plain concrete	1.00
2.88 percent longitudinal reinforcement	1.09
2.88 percent longitudinal reinforcement + 0.49 percent 45 deg spiral reinforcement	2.34
1.44 percent inclined reinforcement	1.27

All photographs of spirally reinforced specimens show a continuous helical fracture at 45 deg to the axis, at right angles to the spiral reinforcement, and a large angle of twist at failure.

5. Kasarnowsky, S., "Design of Statically Indeterminate Reinforced Concrete Structures, Taking Account of the Torsional Moments," *Schweizerische Bauzeitung*, V. 69, 1917. (in German).

A theoretical treatment.

6. Graf, O., and Mörsch, E., "Experiments in Torsion to Explain the Shear Strength of Reinforced Concrete," *Forschungsarbeiten auf dem Gebiete des Ingenieurwesens*, Verein Deutscher Ingenieure (Berlin), 1922. (in German).

The experiments on plain and reinforced concrete specimens were carried out at the laboratories of Stuttgart College of Technology. All specimens were of circular section, 15.77 in. diameter. Longitudinal bars, 45-deg spirals, rings, and rings combined with longitudinal bars were used as reinforcement.

Rings alone, and longitudinal bars alone, were of little value; but a combination of the two (0.62 percent of each) produced a 62 percent increase in the ultimate torque. An additional 0.62 percent of spiral reinforcement increased the ultimate torque by 200 percent.

The concrete was mixed in the proportions 1:2:5, with a water-cement ratio of approximately 0.75.

7. Young, C. R., Sagar; W. L.; and Hughes, C. A., "The Torsional Strength of Rectangular Sections of Concrete, Plain and Reinforced," *Engineering Research Bulletin* No. 3, University of Toronto, 1922.

The tests were carried out on 5 x 5 in. square, and 5 x 10 in. rectangular specimens, the concrete used being a 1:2:4 mix. The average maximum shear stress for plain concrete was 570 psi.

The increase in ultimate torque due to longitudinal reinforcement only was virtually nil, the increase due to 0.79 percent longitudinal plus 1.68 percent spiral reinforcement being 80 percent.

8. Miyamato, T., "Torsional Strength of Reinforced Concrete," *Concrete and Constructional Engineering* (London), V. 22, 1957, pp. 637-647.

There were 86 specimens tested in torsion at the laboratories of the Japanese Home Department. All specimens were of circular cross section, 11.8 in. in diameter, the concrete being of 1:2:4 mix, with a water-cement ratio of 0.68. Reinforcing bars were of mild steel, 0.45 in. in diameter. Tests were carried out at the age of 45 days. (Compressive strength of concrete at 45 days was 1820 psi.) Modulus of rigidity varied from 2,090,000 psi at low loads to 1,740,000 psi at 83 percent of the ultimate torque.

Type of specimen tested	Ultimate torque
Plain concrete	1.00
0.71 percent spiral reinforcement at 45 deg	1.59
0.72 percent spiral reinforcement at 60 deg	1.57
0.75 percent spiral reinforcement at 30 deg	1.47
1.43 percent spiral reinforcement at 45 deg + longitudinal bars	1.66
1.43 percent spiral reinforcement at 60 deg + longitudinal bars	1.66
1.42 percent spiral reinforcement at 45 deg + lateral rings	1.74
1.45 percent spiral reinforcement at 30 deg + lateral rings	1.73
1.43 percent longitudinal bars + lateral rings	1.12
1.22 percent two layers of spiral reinforcement at 45 deg	2.07
2.08 percent two layers of spiral reinforcement at 45 deg + longitudinal bars	2.18

All specimens showed helical cracks at 45 deg to the axis. The resistance of the reinforced test pieces was not exhausted at the maximum torque, and with decreasing load the twist could be further increased. Reinforcing bars were in no case stressed to breaking point. The angle of twist was not affected by the type of reinforcement.

9. Rausch, E., "Design of Reinforced Concrete in Torsion," Dissertation for the degree of Doctor of Engineering, Technische Hochschule, Berlin, 1929. (in German).

Rausch used the data of Bach and Graf[4] in support of a theory for the strength of reinforced concrete in torsion. Several examples are included demonstrating the application of the theory to the design of foundation beams.

10. Emperger, F., editor, *Handbuch fur Eisenbeton*, Wilhelm Ernst and Sons, Berlin, V. 1, 1930, pp. 220-226. (in German).

A note by O. Graf on the torsional strength of concrete, including a brief summary of the experimental work carried out prior to 1930.

11. Turner, L., and Davies, V. C., "Plain and Reinforced Concrete in Torsion," *Selected Engineering Papers* No. 165, Institution of Civil Engineers, (London), 1934.

Six plain concrete beams of square, rectangular, and T-shaped cross section, and six reinforced square beams were tested in torsion at the Battersea Polytechnic Institute. The concrete was mixed in the proportions 1:1½:3. Beams were cured under water, and tested saturated at the age of about 35 days. The quality of the concrete varied considerably, and this affected the torsional strength of some of the beams.

12. Andersen, P., "Experiments with Concrete in Torsion," *Transactions*, ASCE, V. 100, 1935, pp. 949-983.

Six plain concrete specimens of circular cross section, and 42 reinforced specimens, 10 in. *square* by 28 in. long, were tested in torsion at the University of Illinois. High tensile steel, ½-in. diameter, for longitudinal bars, 5/32-in. diameter mild steel *circular* hoops, and 5/32-in. diameter mild steel *circular* 45-deg spirals were used as reinforcement, separately and in combination. Hollow circular plain concrete cylinders were tested in torsion at constant load for a period of 110 days. The control cylinder strength varied from 1950 to 5535 psi.

The principal tensile and compressive strains in the concrete were measured with Huggenberger tensometers using steel plugs grouted into the concrete. A noticeable difference was observed between the tensile and compressive strains as recorded by the extensometers. As the principal stresses must be numerically equal, Andersen assumed that either the value of the modulus of elasticity, or the value of Poisson's ratio differed in tension and compression.

The modulus of rigidity varied from 1,420,000 psi for a cylinder strength of 2000 psi (ultimate torsional strength 250 psi) to 2,140,000 psi for a cylinder strength of 5200 psi (torsional strength 420 psi); these are secant moduli at ¾ of the ultimate strength.

A formula for the torsional strength of reinforced concrete is derived from theoretical considerations.

13. Gilkey, H. J., Discussion of Reference 12, *Proceedings*, ASCE, V. 61, 1935, pp. 131-141.

Reference is made to a report by H. J. Gilkey and F. Vogt, "Tests of Models of Arch Dams and Auxiliary Concrete Tests Conducted by the Bureau of Reclamation at the University of Colorado," Engineering Foundation, Committee on Arch Dam Investigation, Subcommittee on Model Tests, V. 2, May 1934.

A large number of plain concrete specimens were tested in tension, compression, bending, and torsion. The torsion specimens were 3-in. diameter cylinders 12 in. long. They were cured under a variety of conditions and tested at different ages, some wet and some dry. If computations are based on the elastic theory, the ratio of compressive to torsional strength varied from 6 to 11, the ratio of flexural to torsional strength from 1.3 to 1.5, and the ratio of tensile to torsional strength from 0.6 to 0.8, Poisson's ratio varied from 0.15 to 0.25, with an average value of 0.20. Young's modulus varied from 1,830,000 psi for a crushing strength of 1780 psi to 3,320,000 psi for a crushing strength of 4850 psi.

14. Jakobsen, A., "Torsional Moments in Girder Bridges with Two Main Girders," *Beton und Eisen* (Berlin), V. 34, 1935, pp. 242-246. (in German).

An extension of the earlier work of Kasarnowsky.[5]

15. Andersen, Paul, "Rectangular Concrete Sections Under Torsion," ACI JOURNAL, V. 9, No. 1, Sept.-Oct. 1937 (*Proceedings* V. 34), p. 1-11.

Further discussion of Andersen's earlier work.[12] Some further experiments carried out at the University of Illinois are included. It is shown that the ratio of torsional strength to compressive strength decreases as the compressive strength increases.

16. Ruchadze, A., "Torsion and Deformation by Shear Forces of an Elastic Beam, Consisting of Two Different Materials with Epitrochoidal Boundaries," *Travaux de l'Institut Mathematique de Tbilissi*, 1937, pp. 125-139. (in Georgian, with Russian summary).

An extension of St. Venant's theory of torsion. It is claimed that the method can be applied to the design of reinforced concrete.

17. Andersen, P., "Design of Reinforced Concrete in Torsion," *Transactions*, ASCE, V. 103, 1938, pp. 1503-1526.

Contains a further discussion of Andersen's formula[12] as applied to rectangular sections. The distribution of torsional moments in space frames is analyzed by the Hardy Cross method of moment distribution.

18. Kopciowski, J., "Torsion in Reinforced Concrete," *Travaux* (Paris), V. 23, 1939, pp. 69-75. (in French).

The paper is intended to provide practical information on the design of reinforced concrete members in torsion. Formulas for the determination of the stresses arising from torsional moments are derived for various types of cross section.

19. Marshall, W. T., and Tembe, N. R., "Experiments on Plain and Reinforced Concrete in Torsion," *Structural Engineer* (London), V. 19, 1941, pp. 177-191.

There were 26 plain concrete specimens of circular, rectangular, T-shaped, and L-shaped cross section, and 24 rectangular and T-beams with mild steel reinforcement tested at the City and Guilds College, London.

The concrete was mixed in the proportions 1:2:4, with a water-cement ratio of 0.55. The average modulus of rigidity for the rectangular specimens was found to be 2,400,000 psi. Allowing for the effects of shrinkage and plastic yield, a value of 1,200,000 psi is recommended.

Plain rectangular specimens showed no cracks at the center of the long sides prior to the formation of an instantaneous crack at failure.

Rausch's spiral reinforcement formula gave consistent values for the steel stress at failure, although the experiments showed that the steel stress at failure was nowhere above the yield point, and failure appeared to be caused by a breakdown of bond.

20. Marshall, W. T., "The Torsional Resistance of Plastic Materials with Special Reference to Concrete," *Concrete and Constructional Engineering* (London), V. 34, 1944, pp. 83-88.

Previous experimental work by a number of investigators is analyzed to show that consistent values for the ultimate torsional shear stress are obtained if concrete is assumed to be an ideal plastic material.

21. Rao, K., "Ultimate Strength of Concrete Sections in Bending and Torsion by Extension of the Standard Theory," *Structural Engineer* (London), V. 22, 1944, pp. 309-322.

A theoretical treatment based on the assumption that rupture occurs when the layer at the center of gravity of the shear stress diagram reaches the value of the ultimate shear stress.

22. Nylander, H., "Torsion and Torsional Restraint by Concrete Structures," *Meddelanden*, No. 3, Statens Kommitte för Byggnadsforskning, (Stockholm), 1945. (in Swedish, English summary).

Reports tests on about 60 beams carried out at the Royal Technical College, Stockholm. The beams were tested in pure torsion, combined torsion and bending, combined torsion and shear, and combined bending and compression. Subsidiary tests were made to establish the properties of the materials used. In addition, two rigid frames consisting of three parallel beams connected by two cross girders at the ends were tested by applying two concentrated loads to the central beam. Nylander concludes that the stress distribution at failure is such that the maximum shearing stress over the whole cross section has a constant value equal to the tensile strength, i.e., that concrete for the purpose of assessing the ultimate torsional strength may be treated as a plastic material. This is particularly evident from the results of tests on T-shaped sections. It follows that the addition of shearing forces decreases the torsional strength, whereas the addition of direct compression produces an increase in torsional strength. It was found that

bending increased slightly the resistance to torsion. A formula is derived for computing the longitudinal reinforcement required in sections subject to combined torsion and bending, but the design of torsional shear reinforcement is not considered. The paper includes a theoretical discussion of the torsional restraint exercised by two main beams supporting a thin slab at its longer sides. Formulas for the torsional strength of a wide range of sections for ideal elastic and ideal plastic materials are given in an appendix.

23. Nylander, H., "Torsion and Torsional Restraint in Concrete Structures," *Betong* (Stockholm), V. 31, 1946, pp. 16-17. (in Swedish).

Reply to criticisms and comments on Reference 23.

24. Matheson, J. L., "Moment Distribution Applied to Rectangular Rigid Space Frames," *Journal*, Institution of Civil Engineers (London), V. 29, 1948, pp. 221-243.

Develops a more detailed analysis of the problem previously discussed by Andersen.[17]

25. Cuerel, J., "New Waterloo Bridge at London," *Preliminary Publication*, Third Congress, International Association for Bridge and Structural Engineering, Liege, 1948, pp. 367-380.

Contains a note on experiments on the torsional strength of box girders carried out in connection with the design of the bridge.

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Title No. 56-37

Effect of Shear on Ultimate Strength of Rectangular Beams with Tensile Reinforcement

By GEOFFREY BROCK

The effect of shear on the ultimate moment of resistance of rectangular beams was investigated with the aid of small scale reinforced plaster beams. The modes of failure are defined and classified.

A hypothesis is developed that the effect of shear simply reduces the potential moment of resistance below that which would be developed in pure flexure. This hypothesis forms the basis of a simple method of predicting the ultimate load and mode of failure from the known bending moment and shearing force diagrams for any beam. It is checked by the results of experiments on beams under distributed load.

Results of the experiments are compared, where possible, with those obtained on reinforced concrete beams and published by other investigators.

THE ULTIMATE RESISTANCE OF A REINFORCED concrete beam to a uniform bending moment can be predicted by well-established ultimate load design methods.[1] However, when such a beam is also subjected to shearing forces (variations of bending moment) it may fail in other ways. These other modes of failure are usually described as shear failures, but at present there are no criteria for deciding whether a given beam will fail in shear before it fails in bending, or, if so, what the reduction in its load capacity will be. This is a serious hindrance to the application of ultimate load design to reinforced concrete framed structures.

There have been many laboratory investigations devoted to the study of shear failures in reinforced concrete beams.[2] Most investigators have sought to establish limiting values of the shearing stress at failure, calculated from the classical formula of Morsch, $q = V/bjd$. Recently attention has been paid to the possibility that the criteria for such failures may involve limiting moments rather than limiting shear stress.[3]

ACI member Geoffrey Brock is lecturer in civil engineering at University of Birmingham, Birmingham, England, where he has initiated the study of reinforced models of which this paper describes a part. Previously Dr. Brock had engaged in a variety of civil engineering work in England and New Zealand, being especially concerned with the design and construction of prestressed concrete bridges.

The difficulties involved in making and testing large numbers of reinforced concrete beams at research establishments have forced most investigators to base their conclusions on a limited series of tests or to assemble data from tests made by others. This is not the most satisfactory way to investigate such a variable material as reinforced concrete, because a few erratic results can be quite misleading. An ideal experimental investigation of these problems would require a large number of tests under controlled conditions with duplications to establish how far each result is reproducible. Such an investigation, even on beams of moderate size, would be time-consuming and expensive.

The writer has already described how reinforced plaster models can be used to simulate the ultimate load behavior of reinforced structures.[4] Reinforced beams of this kind can be produced in large numbers and tested to destruction relatively easily, and probably with less random variation than reinforced concrete beams. They are, therefore, a useful medium for investigating the effect of shear on ultimate strength and should provide a valuable indication of the way in which reinforced concrete beams behave.

This paper will only refer to the behavior of beams without web reinforcement. Such an examination is of practical value in that it will indicate the parts of beams which are in need of such reinforcement. A systematic investigation of the effectiveness of different kinds of shear reinforcement may then be possible.

Notation

a = shear span of beam under two symmetrical point loads

A_s = cross-sectional area of tensile reinforcement

b = breadth of rectangular beam

d = effective depth of rectangular beam

f = crushing strength of 1-in. plaster cube

f_c' = crushing strength of 6 x 12-in. concrete cylinder

f_y = yield stress of reinforcement, or 0.2 percent proof stress of reinforcement without pronounced yield

M = bending moment

M_u = ultimate bending moment

p = proportion of tensile reinforcement ($=A_s/bd$)

p_b = proportion of reinforcement required for "balanced" failure

V = shearing force

TESTS ON REINFORCED PLASTER BEAMS

These beams are made from alpha hemihydrate gypsum plaster, reinforced with screwed steel rod of about 1/12 in. diameter. Such reinforcement bonds to plaster to about the same extent as deformed bars do to concrete. Because this kind of plaster sets within about 10 min of mixing with water, specimens can be tested conveniently at an age of 1 hr. The crushing strength of the plaster, which is measured on 1-in. cubes, can be closely controlled by the plaster-water ratio employed. Consequently the crushing strength of the plaster in a particular test can be arranged in advance. Some of the more important characteristics of the materials are summarized in the appendix.

More complete details of the technique are given in Reference 4, together with the results of an investigation into the flexural strength of reinforced plaster beams. They showed that the ultimate flexural resistance determined by experiment agreed closely with that predicted by the equations which Whitney[5] successfully established for reinforced concrete beams. A slight modification of his empirical constants allows the 1-in. cube strength of plaster to be substituted for the 12 x 6-in. cylinder strength of concrete. These modifications are shown in Fig. 1, the equation for tension failures being $M_u/fbd^2 = 0.407\ p/p_o\ (1 - 0.24\ p/p_o)$.

Experiments on beams under two point loads

A simple arrangement for investigating the effect of shearing forces on the flexural resistance of a beam is that shown in Fig. 2. Just outside the loading points C and D the maximum moments and shears act on the same critical sections at one of which the beam will ultimately fail. The ratio of moment to shear at this section is simply the shear span a.

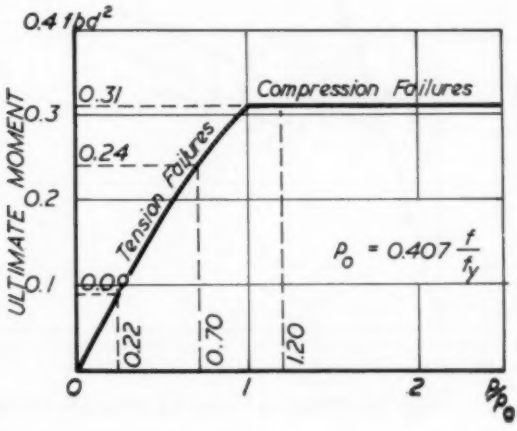

Fig. 1—Whitney curves for flexural failures modified for reinforced plaster beams

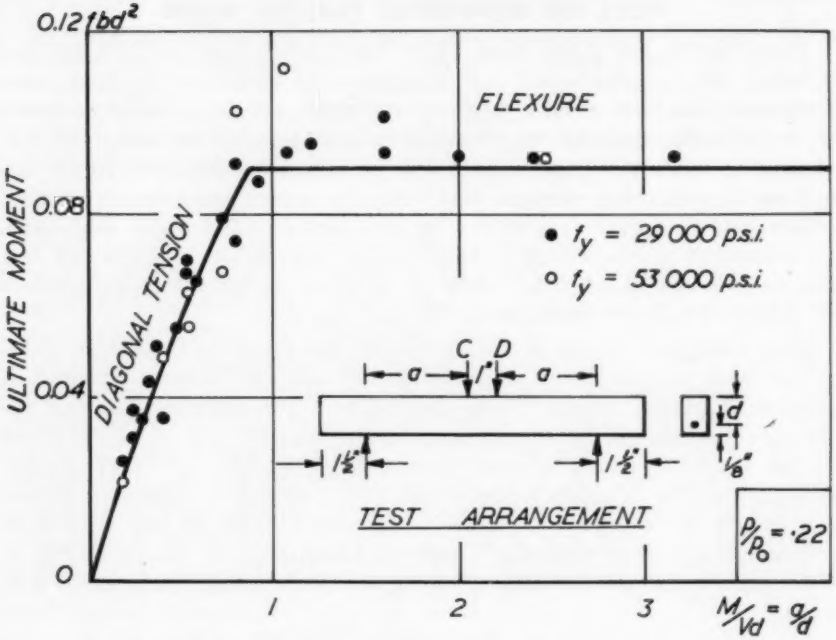

Fig. 2—Effect of shear on ultimate flexural resistance, Series I

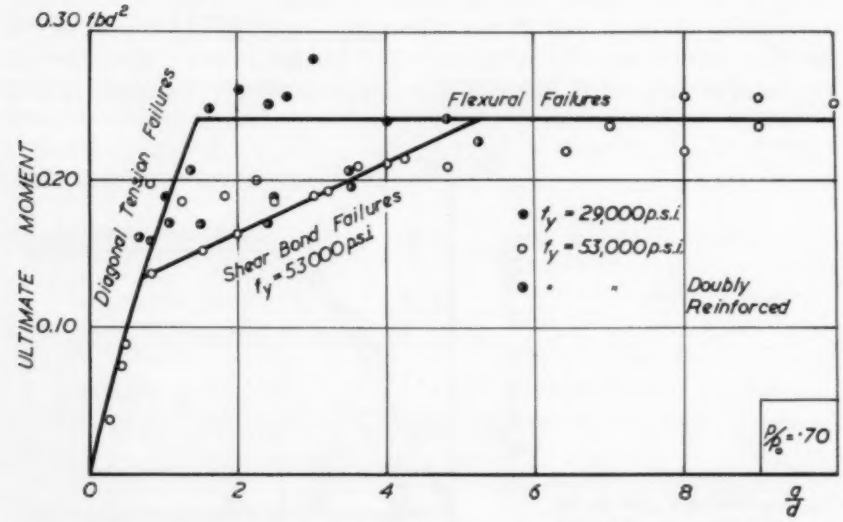

Fig. 3—Effect of shear on ultimate flexural resistance, Series II

By testing beams of the same cross section with different shear spans it is possible to determine the effect of the M/V ratio on the ultimate moment at C.

Reverting to Fig. 1, it will be noted that for purely flexural failures (large M/V), the ultimate moment expressed in terms of fbd^2 depends on the value of p/p_o. Tests made to establish the effect of smaller M/V values should therefore be made with p/p_o held constant.

Three such series of tests have been made at values of p/p_o of 0.22, 0.70, and 1.20. Details of each beam tested are given in Tables 1, 2, and 3. Values of the ultimate moment at C, expressed in terms of fbd^2, have been plotted against M/Vd $(= a/d)$ in Fig. 2, 3, and 4. The fixed dimensions of the test beams are indicated in Fig. 2, sufficient overhang beyond the supports being provided to prevent anchorage failures. It will be found convenient to examine the results of these tests in two groups.

Experiments with $p/p_o = 0.22$ — The ultimate moment decreases rapidly when a/d falls below 0.9, but the shears associated with higher values of a/d appear to have little effect on the ultimate flexural resistance. Fig. 2 shows that the failures divide into two well defined classes, diagonal tension and flexure. The typical appearance of each type of failure is shown in Fig. 5.

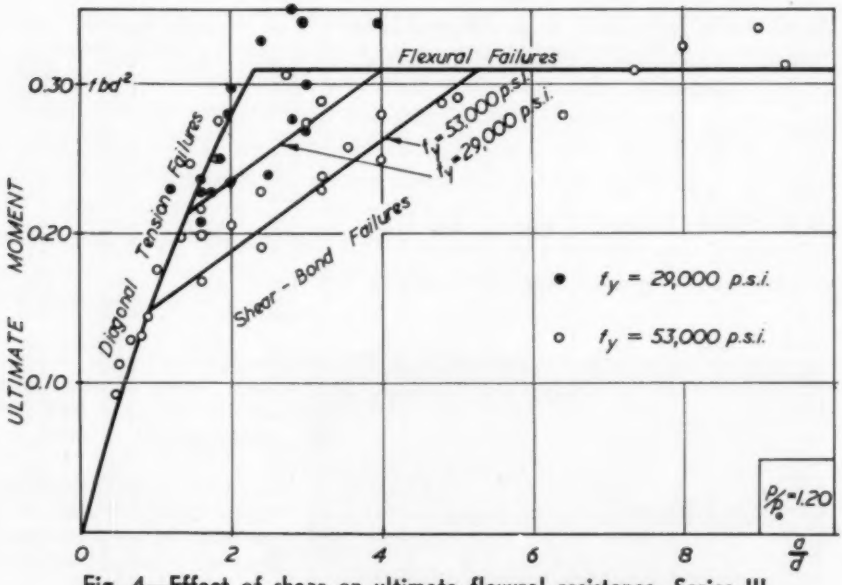

Fig. 4—Effect of shear on ultimate flexural resistance, Series III

Diagonal tension failures occur suddenly by the formation of a crack which runs from the load point to the adjacent support. Careful observation under slow loading indicates that the crack forms at the mid-height of the beam and progresses rapidly in each direction. As soon as the crack forms, the load capacity of the beam falls sharply; failure is instantaneous.

Flexural failures are similar to those in pure bending and, with such lightly reinforced beams, are gradual. The effect of shear on flexural failures produces cracks which curve over toward the top of the beam instead of remaining vertical. Such curvature increases with the decrease of a/d. The values of M_u/fbd^2 for flexural failures are close to that predicted from Fig. 1.

Experiments with $p/p_o = 0.70$ and 1.20 — Some of these tests followed the general pattern of those just described, but others revealed a third mode of failure, also shown in Fig. 5. It is termed "shear-bond" failure, because it involves the destruction of the layer immediately above the reinforcement. The effect is shown clearly in Fig. 6 where it has caused complete separation of the beam without disturbing the plaster beneath the reinforcement, which remains uncracked. Should the reinforcement be concentrated in the interior of a beam, this type of failure

TABLE 1—DETAILS OF BEAMS OF SERIES I

(p/p_o approximately 0.22)

b, in.	d, in.	f_y, psi	f, psi	Reinforcement	a, in.	p/p_o	M_u/fbd^2	a d
0.66	1.375	29,000	2450	2 No. 10 BA	0.50	0.224	0.051	0.36
—	—	—	—	—	0.25	—	0.026	0.18
—	—	—	—	—	0.80	—	0.065	0.58
—	—	—	—	—	0.33	—	0.037	0.24
0.75	0.625	—	—	1 No. 10 BA	1.00	0.219	0.101	1.60
—	—	—	—	—	0.50	—	0.090	0.80
—	—	—	—	—	0.33	—	0.070	0.53
0.375	—	—	—	—	1.00	0.199	0.079	0.73
—	—	—	—	—	0.66	—	0.055	0.48
0.75	1.375	53,000	4150	2 No. 10 BA	0.75	0.214	0.056	0.54
—	—	—	—	—	1.00	—	0.068	0.73
—	—	—	—	—	0.25	—	0.022	0.18
—	—	29,000	4150	2 No. 8 BA	1.25	0.198	0.087	0.91
—	—	—	—	—	0.33	—	0.032	0.24
—	—	—	—	—	0.40	—	0.036	0.29
0.75	0.625	—	—	1 No. 8 BA	1.25	0.217	0.092	2.00
—	—	—	—	—	1.00	—	0.094	1.60
—	—	—	—	—	2.00	—	0.092	3.16
—	—	—	—	—	0.75	—	0.096	1.20
—	—	—	—	—	0.50	—	0.074	0.80
—	—	—	—	—	0.25	—	0.036	0.40
—	—	53,000	—	1 No.10 BA	0.66	0.236	0.112	1.06
—	—	—	—	—	0.33	—	0.062	0.53
—	—	—	—	—	1.50	—	0.092	2.40
—	—	—	—	—	0.50	—	0.103	0.80
—	—	—	—	—	0.25	—	0.048	0.40
—	—	29,000	—	1 No. 8 BA	1.50	0.217	0.092	2.40
—	—	—	—	—	0.66	—	0.094	0.94
—	—	—	—	—	0.33	—	0.067	0.53
—	—	—	—	—	0.20	—	0.044	0.32

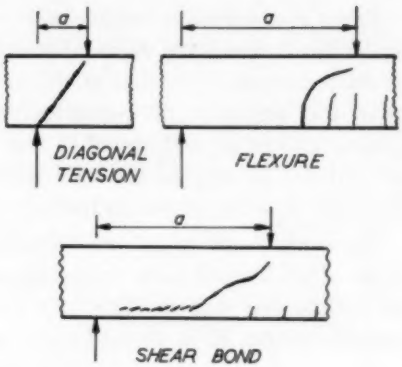

Fig. 5—Typical appearance of three types of failure of beams under a combination of flexure and shear

may show on the soffit as a longitudinal crack beneath one of the bars. Fig. 7 shows such a crack together with the typical cracks on the side of the beam, but the latter do not always form if the soffit has cracked first. In some tests the load could be increased after shear-bond cracks formed, but as any increase was erratic and unreliable, the failing moment was taken to be that which produced the first cracks of this kind.

Shear-bond failures form a transition between flexural and diagonal tension failures. They plot near to straight lines as shown in Fig. 3 and 4. Examination of Table 2 ($p/p_o = 0.70$) reveals that when $f_y = 53{,}000$ psi, shear-bond failures occurred only if f exceeded 3200 psi. No shear-bond failures were observed in this series when f_y was 29,000 psi. Similarly Table 3 ($p/p_o = 1.20$) shows that for $f_y = 53{,}000$ psi, shear-bond failures occurred in tests with f larger than 2060 psi. With $f_y = 29{,}000$ psi they were restricted to values of f not less than 4200 psi.

These results indicate that, for a given value of p/p_o, the occurrence of this mode of failure depends on the values of f and f_y. It is most likely to develop when both these stresses are high and a large amount of reinforcement is used. At given values of p/p_o and f_y the change from flexural to shear-bond failures takes place as f is increased through a certain value. Some subsidiary tests have been made which confirm this (See Fig. 8).

Fig. 6 — Shear-bond failure under central point load. Note that cover beneath reinforcement remains intact

Fig. 7—Variation of shear-bond failure under central point load. Note splitting crack longitudinally beneath reinforcing rod

Effect of compressive reinforcement — In a number of the beams reported in Table 2 compressive reinforcement was provided. The amount was in each case equal to that of the tension reinforcement. The tests showed that the addition of compressive reinforcement to under-reinforced beams makes no difference to the ultimate moment whatever the mode of failure. (Compressive reinforcement would, however, increase the flexural capacity of beams having p/p_o greater than unity.)

Secondary modes of failure — If a beam is loaded to increase its deformation after failure has occurred, the crack patterns of other modes of failure may appear. The true mode of failure can, therefore, be overlooked unless it is immediately recognized. Such recognition can be

TABLE 2—DETAILS OF BEAMS OF SERIES II
($p/p_o = 0.70$)

b, in.	d, in.	f_y, psi	f, psi	Reinforcement	a, in.	$\frac{M_u}{fbd^2}$	a/d	Mode of failure†
0.75	0.39	29,000	2140	1 No. 8 BA	1.50	0.238	4.00	F
—	—	—	—	—	1.125	0.282	3.00	F
—	—	—	—	—	0.75	0.261	2.00	F
—	—	—	—	—	0.375	0.188	1.00	DT
—	—	—	—	—	1.00	0.256	2.66	F
—	0.625	53,000	2340	—	4.00	0.218	6.40	F
—	—	—	—	—	0.50	0.196	0.80	DT
—	—	—	—	—	0.25	0.073	0.40	DT
—	—	—	—	—	0.40	0.160	0.64	DT
—	—	—	—	—*	0.65	0.170	1.04	DT
—	—	—	—	—*	0.85	0.206	1.35	DT
—	—	—	—	—*	0.40	0.158	0.64	DT
—	—	—	—	—*	1.00	0.247	1.60	F
—	—	—	—	—*	1.50	0.250	2.40	F
—	—	—	—	—*	0.50	0.158	0.80	DT
—	—	—	—	—*	3.00	0.240	4.80	F
—	1.375	—	3200	3 No. 8 BA	2.06	0.151	1.50	S-B
—	—	—	—	—	4.12	0.188	3.00	S-B
—	0.875	—	3360	2 No. 8 BA	1.75	0.163	2.00	S-B
—	—	—	—	—	0.75	0.136	0.86	DT
—	—	—	—	—	0.25	0.036	0.28	DT
—	—	—	—	—	2.20	0.185	2.50	S-B
0.50	0.625	—	3520	1 No. 8 BA*	2.19	0.207	3.50	S-B
—	—	—	—	—*	1.50	0.170	2.40	S-B
—	—	—	—	—*	0.937	0.170	1.50	S-B
—	—	—	—	—*	3.44	0.225	5.50	F
—	—	—	—	1 No. 8 BA	0.78	0.184	1.25	DT
—	—	—	—	—	1.41	0.200	2.25	S-B
—	—	—	—	—	2.66	0.213	4.25	S-B
—	—	—	—	—	0.31	0.087	0.50	DT
0.75	0.39	—	3910	—	3.00	0.255	8.00	F
—	—	—	—	—	3.37	0.255	9.00	F
—	—	—	—	—	3.00	0.218	8.00	F
—	—	—	—	—	4.00	0.250	10.70	F
—	—	—	—	—	1.50	0.210	4.00	S-B
—	—	—	—	—	2.62	0.236	7.00	F
—	—	—	—	—	3.37	0.236	9.00	F
—	0.625	—	4690	2 No. 8 BA	2.25	0.208	3.60	S-B
—	—	—	—	—	1.12	0.189	1.80	S-B
—	—	—	—	—	1.00	0.152	1.60	S-B
—	—	—	—	—	2.00	0.192	3.20	S-B
—	—	—	—	—	3.00	0.207	4.80	S-B
—	—	—	—	2 No. 8 BA*	1.56	0.188	2.50	S-B
—	—	—	—	—*	2.18	0.195	3.50	S-B

* Denotes specimen with equal reinforcement in compression.
† F—flexural; DT—diagonal tension; S-B—shear-bond.

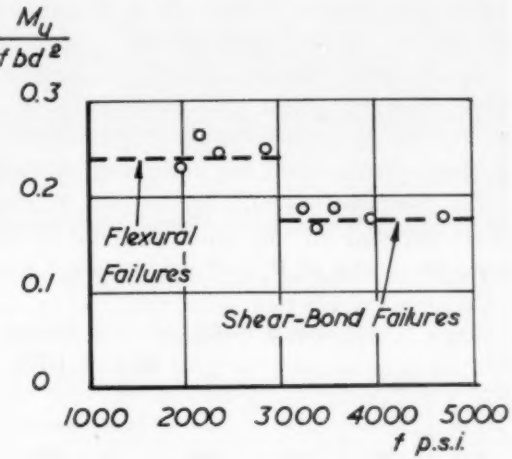

Fig. 8—Effect of variation in plaster strength on mode of failure ($a/d = 2$, $p/p_o = 0.70$, $f_y = 53{,}000$ psi)

difficult when the primary and secondary modes of failure develop almost simultaneously.

Tests on beams loaded at eight points

The tests with two point loads, although useful in determining the effect of shear, are not representative of the kinds of beams used in practical designs. To investigate the effect of more usually distributed loads, tests have been made on simple beams loaded at eight equally spaced points. Three series were run, one at each of the values of p/p_o already investigated. In each the ratio of span to depth was varied by keeping L constant and changing d. The method of test is shown in Fig. 9, and the midspan ultimate moments are plotted against L/d in Fig. 10. We will consider later how these results can be checked in terms of the tests under two point loads.

METHOD OF PREDICTING FAILURE UNDER ANY LOADING

It has been shown that there are two basic modes of shear failure, diagonal tension and shear-bond. Many failures displaying inclined cracks are not shear failures at all. They are merely modifications to the crack patterns of flexural failures which take place without any reduction in the ultimate flexural resistance. True shear failures, on the other hand, always cause a reduction in the flexural capacity, which is a function of the reinforcement index, p/p_o, and the value of M/Vd at the section of failure.

The method of loading under two point loads was useful because the critical section is always just outside one of the loading points. Although the pattern of cracks may extend over a considerable length of the beam, failure always occurs at a crack which reaches the compression

zone at the critical section. With two-point loading the critical section is also that of maximum shear and moment. But for most loadings of simple beams these maxima do not both occur at the same section and the position of the critical section, and hence of the failing crack, will not be known. To predict the position of this section from known shear and moment distributions the following assumptions will be made.

(a) The potential capacity for resisting moment at any section of the beam depends on the value of M/Vd at that section. It can be found from the curve of M_u/fbd^2 against a/d for the appropiate value of p/p_o.

TABLE 3—DETAILS OF BEAMS OF SERIES III
($p/p_o = 1.20$)

b, in.	d, in.	f_y, psi	f, psi	Reinforcement	a, in.	$\frac{M_u}{fbd^2}$	a/d	Mode of failure*
0.50	0.625	53,000	2060	1 No. 8 BA	1.15	0.275	1.84	DT
—	—	—	—	—	1.70	0.306	2.72	F
—	—	—	—	—	2.00	0.288	3.20	F
—	—	—	—	—	0.90	0.246	1.44	DT
—	—	—	—	—	2.50	0.249	4.00	S-B
—	—	—	—	—	3.12	0.291	5.00	S-B
—	—	—	—	—	1.00	0.199	1.60	DT
—	—	—	—	—	2.50	0.280	4.00	S-B
—	—	—	—	—	1.875	0.274	3.00	F
—	—	29,000	2250	2 No. 8 BA	1.00	0.228	1.60	DT
—	—	—	—	—	1.15	0.250	1.84	DT
0.75	—	—	—	3 No. 8 BA	1.00	0.208	1.60	DT
—	—	—	—	—	1.75	0.350	2.80	DT
—	—	—	—	—	1.25	0.298	2.00	DT
—	—	—	—	—	1.50	0.330	2.40	DT
—	1.125	53,000	2290	—	2.25	0.206	2.00	S-B
—	—	—	—	—	4.00	0.258	3.56	S-B
—	—	—	—	—	1.00	0.144	0.89	DT
—	—	—	—	—	1.50	0.190	1.33	DT
—	—	—	—	—	0.75	0.128	0.66	DT
—	—	—	—	—	0.50	0.090	0.45	DT
—	0.39	—	2340	1 No. 8 BA	3.50	0.314	9.35	F
—	—	—	—	—	2.75	0.310	7.35	F
—	—	—	—	—	1.80	0.288	4.80	S-B
—	—	—	—	—	3.00	0.326	8.00	F
—	—	—	—	—	3.38	0.338	9.00	F
0.50	0.50	29,000	2520	3 No. 10 BA	1.00	0.278	2.00	DT
—	—	—	—	—	1.50	0.342	3.00	F
—	—	—	—	—	2.00	0.342	4.00	F
0.75	0.625	53,000	2750	2 No. 8 BA	1.50	0.227	2.40	S-B
—	—	—	—	—	1.00	0.216	1.60	DT
—	—	—	—	—	0.50	0.131	0.80	DT
—	—	—	—	—	2.00	0.228	3.20	S-B
—	—	—	4150	3 No. 8 BA	1.50	0.191	2.40	S-B
—	—	—	—	—	2.00	0.238	3.20	S-B
—	—	—	—	—	0.50	0.132	0.80	DT
—	—	—	—	—	1.00	0.168	1.60	S-B
—	—	—	—	—	4.00	0.280	6.40	F
—	—	—	—	—	0.625	0.176	1.00	DT
—	—	—	—	—	0.312	0.112	0.50	DT
0.5	0.5	29,000	4200	2 No. 8 BA	1.25	0.238	2.50	S-B
—	—	—	—	—	1.50	0.270	3.00	S-B
—	—	—	—	—	1.00	0.234	2.00	S-B
—	—	—	—	—	1.50	0.300	3.00	S-B
—	—	—	—	—	0.80	0.228	1.60	S-B
—	0.625	—	6050	3 No. 8 BA	1.00	0.235	1.60	S-B
—	—	—	—	—	1.75	0.278	2.80	S-B
—	—	—	—	—	0.75	0.230	1.20	DT

*F—flexural; DT—diagonal tension; S-B—shear-bond.

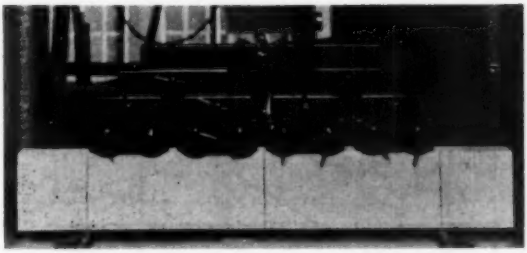

Fig. 9—Method of testing reinforced plaster beam under eight equal point loads

(b) The critical section of the beam will be that at which the actual bending moment first reaches its capacity value, and the ultimate load will be that which produces the capacity moment at the critical section.

The application of these principles will best be demonstrated by the following example.

Beam loaded by eight point loads

A curve showing the variation of M/Vd across the span can be prepared from the known bending moment and shearing force diagrams. This curve is shown, in terms of L/d, on the right of Fig. 11. If we know the value of p/p_o, in this case assumed to be 0.70, and whether or not

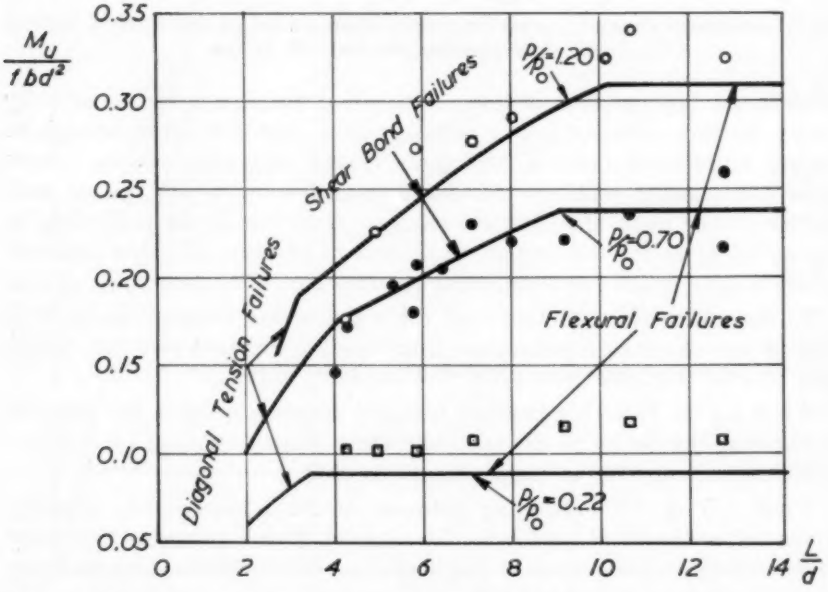

Fig. 10—Simple beam under eight point loads. Variation of midspan moment at failure with L/d ratio for various values of p/p_o. (f_y = 53,000 psi; f chosen to produce shear-bond failures where possible)

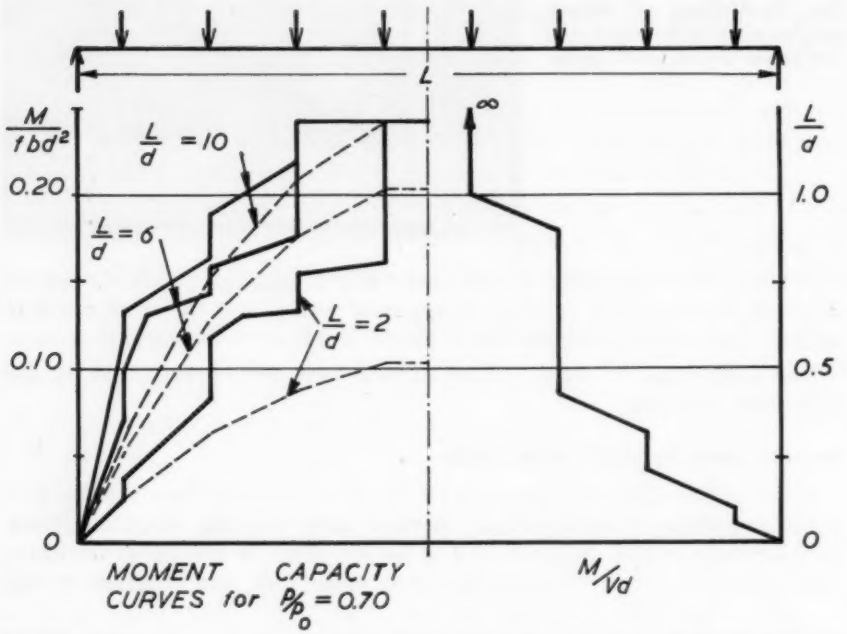

Fig. 11—Moment capacity curves for simple beam loaded at eight points. Dashed lines show bending moments at failure

shear-bond failures can develop, we can draw the moment capacity curve. In this example it has been assumed that f is large enough to permit shear-bond failures, and three typical moment capacity curves have been drawn, with the aid of Fig. 3, on the left of Fig. 11. At each section of the beam the ultimate moment from Fig. 3, corresponding to the M/Vd value for the section, is plotted to give the moment capacity. There will, of course, be a different capacity curve for each value of L/d.

To determine the ultimate load for a particular capacity curve it is merely necessary to superimpose that bending moment diagram which just touches but does not cross the capacity curve.

When $L/d = 10$ such a bending moment diagram touches the capacity curve near the center of the span and the midspan moment reaches the full flexural capacity of the section, $0.24\ fbd^2$.

When $L/d = 6$ the bending moment diagram touches the capacity curve under the third load from the support. This represents a moment of $0.175\ fbd^2$, and hence from Fig. 3 a shear-bond failure. The midspan moment can only reach $0.205\ fbd^2$.

When $L/d = 2$ the failing bending moment diagram coincides with the capacity diagram from the support to the first load point. A diagonal

tension failure may be expected and the midspan moment will only reach 0.105 fbd^2.

This procedure has been used to construct the curves drawn in Fig. 10 showing the ultimate moment to be expected at midspan for a given value of L/d. The experimental results are sufficiently close to these curves to suggest that the procedure just described is suitable for design purposes.

It will be noted that the lightly reinforced beams with $p/p_o = 0.22$ do not suffer any deterioration of their flexural capacity even when L/d

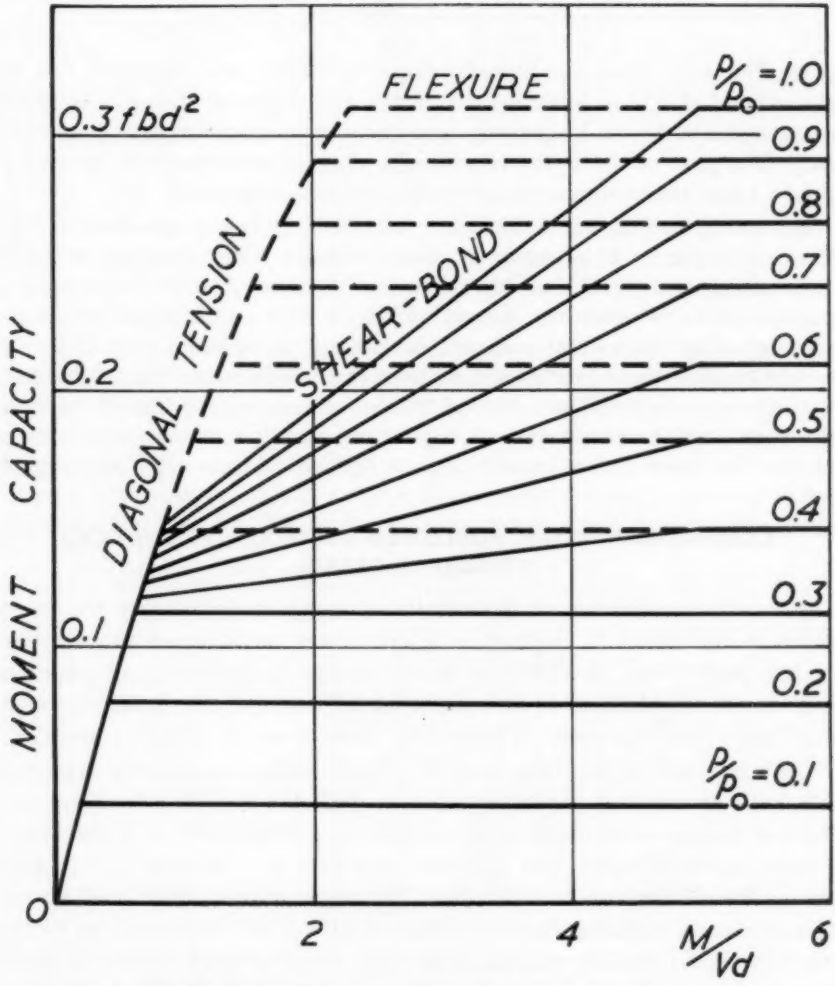

Fig. 12—Generalized design chart for ultimate moment capacity

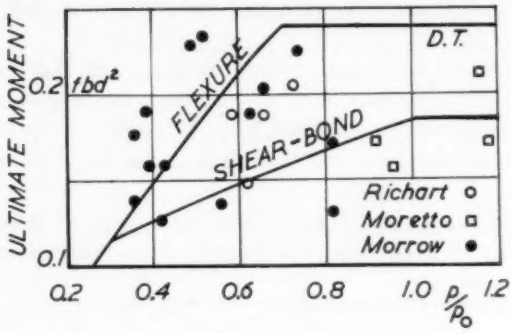

Fig. 13—Results of tests on reinforced concrete beams ($a/d = 1.75$)

is as low as 4. Over-reinforced beams, however, will begin to fail in shear-bond if L/d is less than 10. If the full flexural strength is to be maintained at lower L/d ratios some shear reinforcement will be necessary. The part of the beam requiring such reinforcement can be established from the moment capacity diagrams as follows.

Reverting to Fig. 11, if the beam with $L/d = 6$ is to develop its full flexural capacity at midspan it must withstand the bending moment diagram plotted for $L/d = 10$. There is a deficiency in the moment capacity curves between the second and third load points which must be made good by shear reinforcement in that region. A beam with $L/d = 2$ would require shear reinforcement over the whole span. This illustrates that the conventional practice of placing shear reinforcement only in the regions of maximum shear may be unsafe. Spectacular verification of this has been given by a full-scale failure described by Anderson.[6]

COMPARISON WITH PUBLISHED TESTS ON REINFORCED CONCRETE BEAMS

A valuable summary of the results of many investigations has been given in Reference 3. Most investigators have tested beams under one or two point loads, generally as shown in Fig. 2. Sufficient information is given to enable p/p_o to be calculated for each of the tests, although this factor does not seem to have been considered in previous work.

Only one test series, that of A. P. Clark, had a sufficiently constant p/p_o ratio to warrant direct comparison with the results already given. Twelve beams were made with p/p_o varying from 0.303 to 0.372 about a mean value of 0.329. The ultimate moments at a/d from 1.17 to 2.34 varied from 0.096 $f_c'bd^2$ to 0.149 $f_c'bd^2$. The mean value of 0.131 $f_c'bd^2$ corresponds closely with the flexural value of 0.137 $f_c'bd^2$ calculated by Whitney's method. (The low value of 0.096 $f_c'bd^2$ seems a freak result, the next lowest being 0.122 $f_c'bd^2$. These results confirm those of Fig. 2, showing

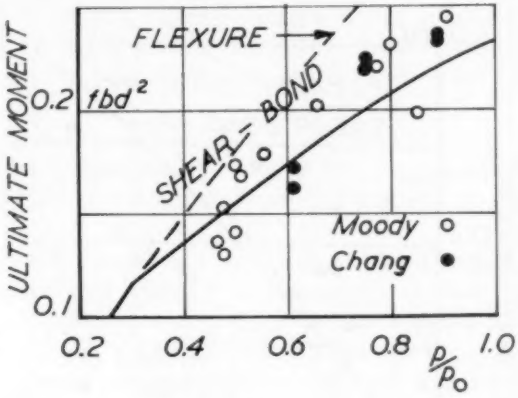

Fig. 14—Results of tests on reinforced concrete beams ($a/d = 3.0$)

that, with small values of p/p_o, a/d must fall below unity before there is any reduction in the flexural capacity.*

Most other tests under similar load arrangements have been made at fixed values of a/d, but with varying values of p/p_o caused by variations in the concrete strength and the proportion of reinforcement. To compare these results with those obtained from plaster beams it is necessary to deduce curves for M/fbd^2 against p/p_o for different a/d values. This has been done by preparing the idealized family of curves for ultimate moment against M/Vd, shown in Fig. 12, by interpolating from Fig. 1, 2, 3, and 4. These curves, which would also serve as a generalized design chart for producing moment capacity curves, have been used to prepare those shown on Fig. 13, 14, 15, and 16. The latter have been chosen with values of a/d suitable for comparison with the available experimental results. These have been taken from investigations by

*In comparing these results with those for plaster beams, it has not been thought worthwhile to make the small changes involved in replacing f by f_c which were indicated in Fig. 1.

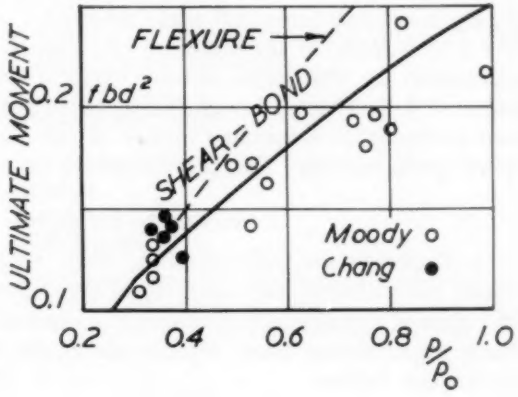

Fig. 15—Results of tests on reinforced concrete beams ($a/d = 3.4$)

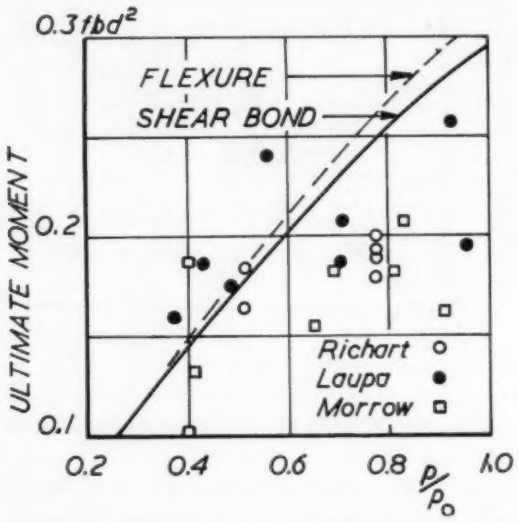

Fig. 16—Results of tests on reinforced concrete beams ($a/d = 4.6$)

K. G. Moody, A. Laupa, F. E. Richart, and O. Moretto collected in Reference 3. Other results given by Morrow and Viest[7] and Chang and Kesler[8] have also been used. There is reasonable agreement between all these results and the curves drawn on the basis of Fig. 12, excepting those on Fig. 16. Here, although the value of a/d is high (4.6) the ultimate moments are on the whole substantially below the almost coincident flexural and shear-bond values. It is thought that these low values were due to anchorage failures. None of the investigations quoted used hooks and the length of the bar carried past the support was relatively less than that allowed on the plaster beams. The latter were found to be quite susceptible to reductions in the anchorage length which produced apparent shear-bond failures.

A series of tests on 18 beams under uniform load has been reported by Bernaert and Siess.[9] Of these all except one had L/d ratios in excess of 8.8. The ratios of the observed ultimate moments at midspan to those calculated by Whitney's method varied from 0.82 to 1.16 with a mean value of 0.98. Thus although there was a great deal of inclined cracking and evidence of secondary modes of failure, the failures of this series were predominantly flexural as would be expected from Fig. 11.

CONCLUSIONS

1. Experiments indicate that, even when flexure is modified by shear, the ultimate load capacity of a reinforced beam can be expressed in terms of the ultimate moment at a critical section (section of failure). Such experiments have been made under conditions which preclude anchorage failure.

2. When a simple beam is loaded at two symmetrically disposed points, the critical section is always at one of them. The crack at which failure occurs reaches the critical section in the compression zone of the beam.

3. The effect of shear on the value of M_u/fbd^2 at the critical section depends on the dimensionless quantities p/p_o and M/Vd. In this relationship all the important variables are included, namely b, d, p, f, f_y, M, and V.

4. The value of M_u/fbd^2 is not diminished by small shears although they may induce patterns of inclined cracks. It is proposed that, despite the appearance of failure, all failures which are not characterized by a reduction of flexural strength be treated as flexural failures.

5. Shear failures are associated with a reduction of the flexural resistance at the critical section. The two basic modes are diagonal tension and shear-bond.

6. Diagonal tension failures occur only when the shear span: effective depth ratio is small. They are virtually independent of the amount of normal tension reinforcement.

7. The development of shear-bond failures depends to a large extent on the reinforcement index, p/p_o. If the latter value is less than about 0.3, this mode of failure does not develop. At values above 0.3, it develops only if the values of f and f_y exceed certain minima.

8. Provided that the value of p/p_o does not exceed unity, the provision of compression reinforcement does not affect the ultimate moment in any mode of failure. For this reason it seems incorrect to describe shear failures as compression failures.

9. A design method has been indicated for predicting the ultimate load and mode of failure. It depends on the distribution of the ratio M/Vd throughout the structure. For determinate structures it is straight forward, but a knowledge of the redistribution of moments at failure would be necessary for continuous structures.

10. The principles given above are in accordance with much of the published evidence relating to reinforced concrete.

ACKNOWLEDGMENT

Credit is due to S. Harrison for assistance with many of the experiments on reinforced plaster beams.

REFERENCES

1. ACI-ASCE Committee 327, "Ultimate Strength Design," ACI JOURNAL, V. 27, No. 5, Jan. 1956 (*Proceedings* V. 52), pp. 505-524.

2. Hognestad, E., "What Do We Know About Diagonal Tension and Web Reinforcement in Concrete?" *Circular Series* No. 64, University of Illinois Engineering Experiment Station, 1952, 47 pp.

3. Laupa, A.; Siess, C. P.; and Newmark, N. M., "Strength in Shear of Reinforced Concrete Beams," *Bulletin* No. 428, University of Illinois Engineering Experiment Station, 1955, 73 pp.

4. Brock, G., "Direct Models as an Aid to Reinforced Concrete Design," *Engineering* (London), No. 4857, Apr. 1959.

5. Whitney, C. S., "Plastic Theory of Reinforced Concrete Design," *Transactions*, ASCE, V. 107, 1942, p. 251.

6. Anderson, Boyd G., "Rigid Frame Failures," ACI JOURNAL, V. 28, No. 7, Jan. 1957 (*Proceedings* V. 53), pp. 625-636.

7. Morrow, JoDean, and Viest, I. M., "Shear Strength of Reinforced Concrete Frame Members Without Web Reinforcement," ACI JOURNAL, V. 28, No. 9, Mar. 1957 (*Proceedings* V. 53), pp. 833-869.

8. Chang, Tien S., and Kesler, Clyde E., "Static and Fatigue Strength in Shear of Beams with Tensile Reinforcement," ACI JOURNAL, V. 29, No. 12, June 1958 (*Proceedings* V. 54), pp. 1033-1057.

9. Bernaert, S., and Siess, C. P., "Strength in Shear of Reinforced Concrete Beams Under Uniform Load," *Civil Engineering Studies, Structural Research Series* No. 120, University of Illinois, 1956, 63. pp.

APPENDIX — PROPERTIES OF MATERIALS USED IN REINFORCED PLASTER BEAMS

The stress-strain characteristics of threaded steel rod and the plaster are given in Fig. A-1, variation of plaster strength with mix proportions in Fig. A-2, and the bond characteristics of threaded steel rod in plaster in Fig. A-3.

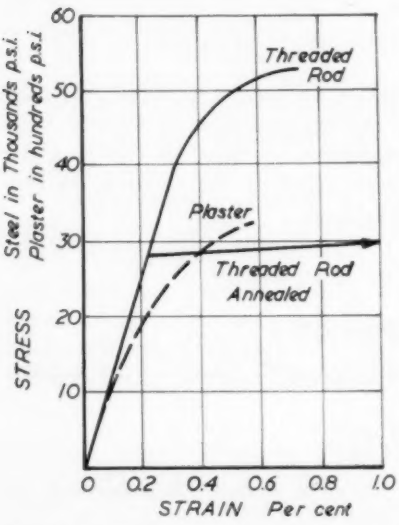

Fig. A-1—Stress-strain curves for plaster (35 percent water) and threaded steel rod (based on full diameter)

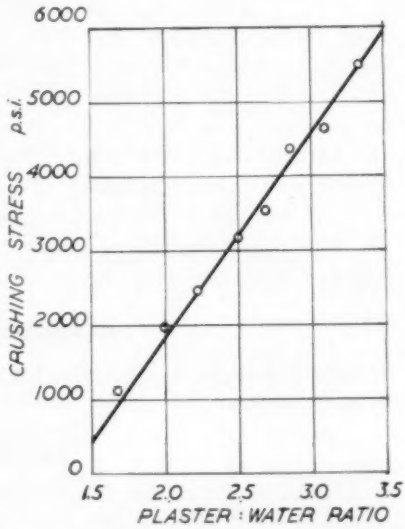

Fig. A-2—Variation of plaster strength with mix proportions. Based on 1-in. cubes tested 1 hr after mixing

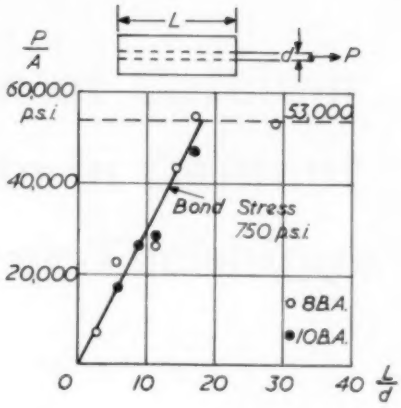

Fig. A-3—Pull-out tests on threaded rods (f_y = 53,000 psi) embedded in plaster (35 percent water) tested 1 hr after casting

TABLE A-1—DIMENSIONS OF BRITISH ASSOCIATION SCREW THREADS

Size (BA)	Pitch, in.	Full diameter, in.	Core diameter, in.
8	0.0169	0.087	0.066
10	0.0138	0.067	0.054

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Title No. 56-38

Determination of Calcium Sulfoaluminate in Cement Paste by Tracer Technique

By TOSHIO MANABE and NAOYA KAWADA

To determine the rate of combination of gypsum with aluminate phases during the first few minutes of initial hydration period, a method using radioactive isotope sulfur-35 as a tracer was studied. The method consists of a combination of Forsen's extraction method with an isotope dilution method. Procedures are described and results discussed. From these results some considerations regarding the initial hydration process of portland cement are presented.

■ IN THE MANUFACTURE OF PORTLAND CEMENT, gypsum is generally added to regulate the setting time. The mechanism through which gypsum controls the setting of cement is not clear and is still under debate. However, it is believed that gypsum is combined with calcium aluminate in double salts, i.e., calcium sulfoaluminates, during the initial hydration period and such formation of double salts plays an important role in the setting process.

Forsen[1] determined the amount of combined gypsum in set cement by dissolving the free gypsum with saturated lime solution, in which the calcium sulfoaluminates are insoluble, and found that all the gypsum in cement was rapidly consumed. But this method is not to be utilized in determination of the amount of gypsum combined during the first few minutes of initial hydration period, because a further combination takes place during the course of extraction.

Recently, the availability of radioactive isotopes as tracers has offered new methods for studies of chemical reactions. To determine the rate of combination of gypsum with aluminate phases during the first few minutes of initial hydration period, a newly developed method using

Toshio Manabe and **Naoya Kawada** are research members of the Research Laboratory of Nihon Cement Co., Ltd., Tokyo, Japan. Their work has included a study of the initial hydration process of portland cement; especially the mechanism of false set and the influences of additives have been investigated.

radioactive isotope sulfur-35 as a tracer was studied. The principle of the method and some results obtained were reported briefly in the discussion of Hansen's ASTM paper.[2]

PRINCIPLE OF METHOD

In brief, the new method consists of a combination of Forsen's extraction method with an isotope dilution method.

Dry cement or cement paste is extracted with lime solution containing SO_3 labeled with sulfur-35. The amounts of SO_3 in the lime solution before and after extraction are analyzed, and the specific radioactivities of SO_3 in both solutions are also counted.

Now, let W_c (mg) represent the total amount of SO_3 in the cement used for the test, W_b (mg) the amount of SO_3 combined as sulfoaluminate in dry cement or in cement paste prior to test (and hence the unknown that is to be determined), W_e (mg) the amount of SO_3 in the lime solution before extraction, W_p (mg) the amount of SO_3 combined as sulfoaluminate during the test, and W_f (mg) the amount of SO_3 in the final solution after extraction. Therefore, the material balance is given by

$$W_e + W_c = W_b + W_p + W_f \quad \text{.....(1)}$$

When the dry cement or paste is extracted with such a lime solution, SO_3 in other forms, namely $(W_c - W_b)$, is dissolved, but at the same time some SO_3, (W_p), forms sulfoaluminate during the test and carries radioactivity with it. Then, the balance of specific activities may be represented by

$$A_e \cdot W_e = A_f \cdot W_f + A_m \cdot W_p \quad \text{.....(2)}$$

where, A_e and A_f are specific activities of SO_3 in the lime solutions before and after extraction, respectively, and A_m is the mean specific activity of SO_3 combined as sulfoaluminate during the test.

It is difficult to determine the true value of A_m, so some assumption must be made.

If it is assumed that SO_3 which has not been combined as sulfoaluminate before test is dissolved instantaneously when cement contacts with water, the activity of SO_3 in sulfoaluminate formed during the test is

TABLE I — RATE OF SOLUTION OF GYPSUM

Sample No.		Total SO_3 in sample, percent	Extracted SO_3		
			1 min	3 min	5 min
1	(Quartz sand + gypsum) ground in ball mill	1.82	1.78	1.79	1.78
2	(Sample 1 + gypsum) mixed in porcelain mortar	2.71	2.54	2.56	—
3	(Sample 1 + plaster of Paris) mixed in porcelain mortar	2.86	2.72	2.73	—

the same as that of SO_3 in the final solution, or $A_m = A_f$. Hence, from Eq. (1) and (2), the amount of SO_3 combined as sulfoaluminate prior to test, W_b, would be given by

$$W_b = (W_s + W_c) - W_s\,(A_s/A_f) \quad \text{.....(3)}$$

But in practice, time is required to dissolve the gypsum, so it may be more probable that SO_3 in sulfoaluminate formed during test would have the average of specific activities of SO_3 in lime solution before test and in final solution. That is, $A_m = 0.5\,(A_e + A_f)$. In this case, the amount of SO_3 combined as sulfoaluminate, W_b, would be given by

$$W_b = W_s + W_c - W_f - \frac{A_e \cdot W_s - A_f \cdot W_f}{0.5\,(A_s + A_f)} \quad \text{.....(4)}$$

To study the rate of solution of gypsum, three samples were prepared. The first sample was ground from quartz sand and natural gypsum in a laboratory mill to fineness comparable to that of portland cement. The second sample was prepared from the first sample by mixing gypsum in a porcelain mortar, and the third was prepared from the first sample by mixing with plaster of Paris in a porcelain mortar. From each sample 1 g was shaken with 100 ml of lime solution for 1, 3, and 5 min. These suspensions were filtered on a Büchner funnel and the extracts were analyzed for SO_3. The results are given in Table 1.

The data of this table show that about 95 percent of total SO_3 was extracted within 1 min, and especially when the sample was ground in the ball mill, 98 percent of SO_3 was extracted within 1 min. Hence, it appears that gypsum ground to fineness comparable to that of portland cement is dissolved in lime solution fairly rapidly.

Therefore, from these results it may be expected that the true amount of SO_3 combined as sulfoaluminate in dry cement or paste prior to test, W_b, would be larger than W_b derived from Eq. (3) but less than W_b derived from Eq. (4).

EXPERIMENTS

Preparation of gypsum labeled with S-35

Gypsum labeled with S-35 was prepared by mixing $CaCl_2$ solution and $(NH_4)_2SO_4$ solution containing a suitable amount of H_2SO_4 labeled with S-35.

Preparation of lime solution containing SO_3 labeled with S-35

Lime solution which contained about 0.8 g per l of CaO and 0.3 g per l of SO_3 was prepared from saturated lime solution and the S-35 labeled gypsum.

Procedure

One g of cement was shaken for 4 min with 100 ml of lime solution containing SO_3 labeled with S-35. In the case of paste, it was made by mixing 1 g of cement and 0.5 ml of distilled water, and after the prescribed period it was shaken with 100 ml of lime solution for 4 min. Then the slurry was filtered on a Büchner funnel and from 20 ml of the filtrate SO_3 was determined and from 50 ml the specific radioactivity of SO_3 in the filtrate was determined.

In measuring the activity of S-35, which is a soft β-ray emitter, it is necessary to consider the effect of self-absorption. But if the sample is sufficiently thick so that the radiation from the lowest layer is completely absorbed by the upper layers, then self-absorption corrections are not necessary (infinite thickness).

In this study, for the determination of specific activity of SO_3 in solution, a suitable amount of $(NH_4)_2SO_4$ solution was added to the 50 ml of filtrate, then $BaSO_4$ was precipitated as usual. After filtering by using a glass filter apparatus, the sample of $BaSO_4$ was counted with 2π gas flow counter. In such a case, the specific activity, A, of SO_3 in filtrate is given by

$$A = \frac{a+b}{a} \cdot A_s \quad \text{..... (5)}$$

where, A_s is the specific activity of SO_3 in solution diluted with $(NH_4)_2SO_4$, and a and b are the amount of SO_3 (mg) in the original filtrate and in the added $(NH_4)_2SO_4$ solution, respectively.

An example will afford a better understanding of the procedure.

1. The cement used in this example was aerated in the atmosphere at high humidity for 24 hr.

2. This cement contains 2.22 percent SO_3, that is, the amount of SO_3 in 1 g of the cement is 22.2 mg.

3. The lime solution for extraction: SO_3 content was 30.9 mg per 100 ml, (W_s), or 15.4 mg per 50 ml.

$(NH_4)_2SO_4$ solution containing 47.5 mg of SO_3 was added to 50 ml of lime solution, $BaSO_4$ was precipitated, and then its activity was counted with the gas flow counter. Its activity was 3554 counts per min. From Eq. (5), the activity of SO_3 in the original lime solution, (A_s), was calculated as follows:

$$A_s = \frac{15.4 + 47.5}{15.4} \times 3554 = 14{,}525 \text{ counts per min}$$

4. The filtrate: 1 g of the cement was shaken with 100 ml of the above lime solution for 4 min and the slurry was filtered. The 20 ml of filtrate were analyzed for SO_3. The filtrate contained 40.2 mg of SO_3 per 100 ml, or 20.1 mg per 50 ml.

TABLE 2 — AMOUNT OF SO_3 COMBINED AS SULFOALUMINATE IN DRY CEMENT

Cement		A	B	C	D	E	F	G	H	I	J
SO_3 content of cement, percent of total weight		2.22	1.68	1.94	1.80	1.45	1.87	2.04	1.66	1.61	1.88
SO_3 combined as sulfoaluminate, percent of total weight	Eq. (3)	0.31	0.05	0.34	0.20	0.32	0.11	0.04	—0.08	0.01	—0.13
	Eq. (4)	0.50	0.18	0.47	0.38	0.38	0.22	0.15	0.07	0.12	0.02
Unextracted SO_3, percent of total weight		1.32	0.66	0.97	0.88	0.72	0.75	1.02	0.86	0.68	0.69

After addition of $(NH_4)_2SO_4$ solution which contained 39.6 mg of SO_3, to 50 ml of the filtrate, $BaSO_4$ was precipitated and its activity was counted. Its activity was 3490 counts per min. From Eq. (5) the activity of SO_3 in the filtrate, (A_f), was calculated as follows:

$$A_f = \frac{20.1 + 39.6}{20.1} \times 3490 = 10{,}355 \text{ counts per min}$$

5. According to Eq. (3), the amount of SO_3 combined with aluminate, W_b, was calculated as follows:

$$W_b = (W_c + W_e) - W_e(A_e/A_f) = (22.2 + 30.9) - 30.9 \times \frac{14{,}525}{10{,}355} = 9.8 \text{ mg}$$

AMOUNT OF SO_3 COMBINED AS SULFOALUMINATE IN DRY CEMENT

Ten commercial cements (corresponding to Type I cement in the United States) were used in the investigation.

The results are shown in Table 2. In this table, the line indicating SO_3 content of cement (percent), shows the total amount of SO_3 analyzed by the standard method. The lines indicating SO_3 combined as sulfoaluminate (percent), show the amounts of SO_3 determined by the procedure using S-35 and calculated from Eq. (3) and (4), respectively. The last line, unextracted SO_3 (percent), shows the amount of SO_3 obtained by applying Forsen's method. (Forsen's method: Cement is extracted by lime solution and the difference between the total amount of SO_3 and the amount of SO_3 soluble in lime solution, i.e., unextracted SO_3, constitutes the combined SO_3.)

These results show that the amounts of SO_3 combined as sulfoaluminate in dry cements were calculated to be —0.13 to 0.34 percent by Eq. (3) or 0.02 to 0.50 percent by Eq. (4).

As already shown, even if gypsum is pure and finely ground, 2 to 5 percent of it is not extracted by 4 min of shaking with lime solution. Moreover, in the case of commercial cement, some part of the SO_3 in the clinker or natural gypsum rock added could not be easily extracted, whereas the above calculation was made on the assumption that all of

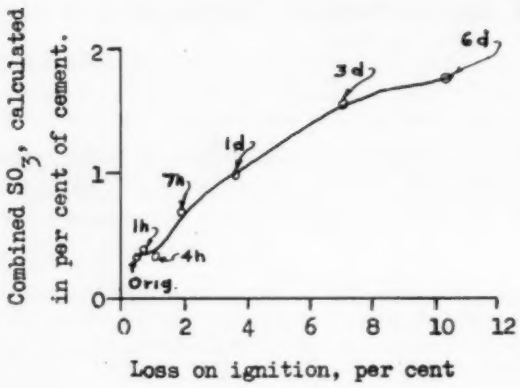

Fig. 1 — Amount of SO_3 combined as sulfoaluminate in cement, aerated under atmosphere of high humidity (relative humidity 90-95 percent). Time indicated along curve shows aeration periods

the SO_3 in cement is extracted after 4 min of shaking with lime solution The amount of unextracted SO_3 would be roughly estimated at 0.2 percent. Consequently, it seems probable that SO_3 combined as sulfoaluminate would be absent in commercial cement.

Thus, this method does not give the true amount of SO_3 combined as sulfoaluminate. However, except for the above, the error caused by this method of procedure is ± 0.05 percent, then the rate of combination of gypsum with aluminate phases during the initial hydration period, which will be referred to later, could be estimated more accurately.

The difference between the amount of SO_3 combined as sulfoaluminate and the unextracted SO_3 indicates clearly that a considerable amount of SO_3 was combined during the course of the shaking.

An attempt was made to determine the amount of SO_3 combined as sulfoaluminate in cement which was aerated in the atmosphere at high humidity (relative humidity, 90 to 95 percent). The results calculated from Eq. (3) are given in Fig. 1. Fig. 1 shows that the combination of gypsum clearly takes place when the loss on ignition of cement reaches about 2 percent or more.

COMBINATION OF GYPSUM DURING THE INITIAL HYDRATION PERIOD

The amounts of SO_3 combined as sulfoaluminate in pastes during the first few minutes of the initial hydration period were determined. The results obtained with five of the commercial cements are given in Fig. 2. In Fig. 2, Cement A, B, and C had normal setting properties, while Cement D and E manifested false setting tendencies. Cement A′ was prepared from Cement A by aeration in the laboratory (room temperature, 20 C; relative humidity, 50 percent), so that it developed false setting tendency. W_b values in Fig. 2 were calculated from Eq. (3).

TABLE 3 — AMOUNT OF SO_3 COMBINED AS SULFOALUMINATE IN PASTE

Period of hydration, min		0	1	2	3	5	7	10
Cement A	From Eq. (3) percent of total weight	0.31	0.75	0.92	1.19	1.26	1.38	1.34
	From Eq. (4), percent of total weight	0.50	0.81	0.96	1.19	1.25	1.38	1.34
Cement B	From Eq. (3), percent of total weight	0.05	0.39	0.48	0.64	0.65	0.68	0.71
	From Eq. (4), percent of total weight	0.18	0.45	0.53	0.66	0.67	0.70	0.72
Cement D	From Eq. (3), percent of total weight	0.20	0.31	0.19	0.52	0.42	1.01	1.04
	From Eq. (4), percent of total weight	0.38	0.41	0.33	0.60	0.51	1.02	1.04

Comparisons between the results derived from Eq. (3) and (4) are given in Table 3 for Cements A, B, and D. The shorter the hydration period, the greater the difference; but after 5 min or so the difference is small. From these results, it may be said that so far as the rate of formation of sulfoaluminate during the initial hydration period is concerned, it makes no great difference whether W_b is calculated from Eq. (3) or (4). Hence, in further discussion the values of W_b calculated from Eq. (3) will be dealt with.

The results in Fig. 2 indicate that in the cases of normal setting cement (A, B, and C), a considerable amount of gypsum is combined as calcium sulfoaluminate within the first few minutes and then the rate of combination becomes very slow. It appears that approximately 50 per-

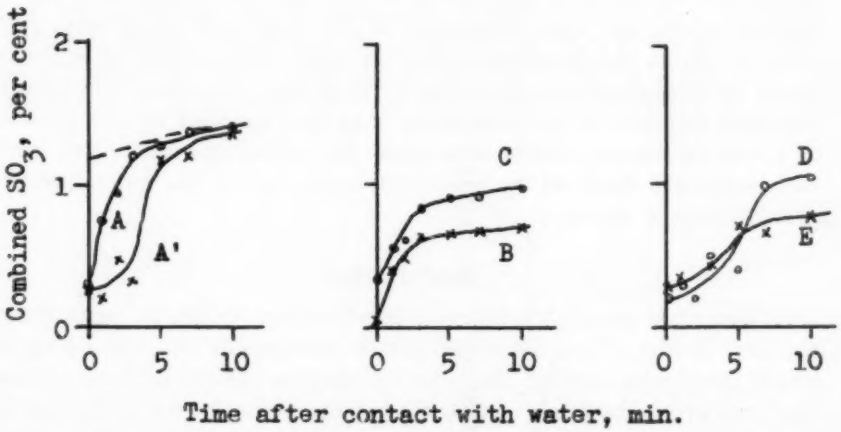

Fig. 2—Amount of SO_3 combined as sulfoaluminate during the first few minutes of initial hydration period. A, B, and C are normal setting cements. A', D, and E are false setting cements

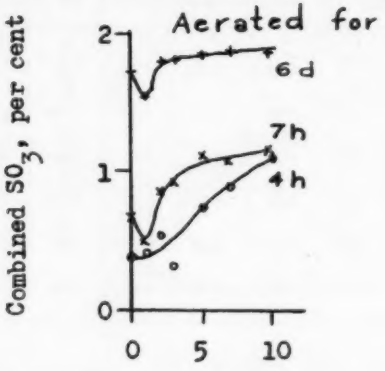

Fig. 3—Amount of SO_3 combined as sulfoaluminate in paste which is prepared from cement aerated in moist air

cent of the total SO_3 in cement pastes may be combined within a few minutes, whereas, in the cases of false setting cements (A′, D, and E), the combination of gypsum takes place about 3 min later; in other words, the combination of gypsum with aluminate is delayed for some reason.

The curve shown by the broken line in Fig. 2 gives the amount of unextracted SO_3 in the paste of Cement A obtained by shaking with lime solution, that is, Forsen's method. This method indicates incorrectly that fairly large percentages of SO_3 are combined as sulfoaluminate during the first few minutes, and continue to be combined at a nearly uniform rate. But at ages greater than about 10 min the results are in substantial agreement with those obtained by the tracer technique.

Fig. 3 illustrates the rates of combination of SO_3 in pastes which were prepared with cements aerated for 4 hr, 7 hr, and 6 days in moist air and containing fairly large amounts of SO_3 combined as sulfoaluminate (Fig. 1). When the cements contact the water the amount of SO_3 combined as sulfoaluminate decreases at first and then increases rapidly. The first decrease of sulfoaluminate may be explained by the presence of alkali carbonates which decompose the sulfoaluminate. After alkali carbonates are depleted by precipitation of $CaCO_3$, the sulfoaluminate will be formed again.

DISCUSSION

As described above, the rates of combination of SO_3 in paste during the first 10 min of the hydration period were easily determined by the newly developed method using the radioactive isotope S-35 as a tracer. The rate of combination of SO_3 during the first 10 min is typified by the curve shown in Fig. 4.

From this curve, the retarding action of gypsum may be explained by the theory of the formation of a protective film of calcium sulfoalumi-

TABLE 4 — AMOUNT OF SO_3 COMBINED AS SULFOALUMINATE IN PASTE AT AGE OF 20 MIN BY FORSEN'S METHOD AND TIME OF SETTING DETERMINED BY VICAT APPARATUS

Sample No.	Total SO_3 content, percent of total weight	Extracted SO_3, percent of total weight	Combined SO_3 as sulfoaluminate, percent of total weight	Time of setting	
				Initial, hr-min	Final, hr-min
1	0.70	0.07	0.64	0-21	0-25
2	0.79	0.08	0.71	3-15	4-40
3	1.00	0.25	0.75	3-25	5-40
4	1.21	0.51	0.70	3-43	5-40
5	1.56	0.84	0.72	3-52	5-40
6	2.13	1.40	0.73	3-50	5-30

nate, as concluded by Forsen and others, and amplified as follows.

Gypsum in cement dissolves rapidly and calcium sulfoaluminate deposits on the surfaces of unhydrated clinker particles as a slightly permeable film ($a \rightarrow b$, in Fig. 4). When the surfaces of aluminate particles are almost covered with the film (b), the subsequent combination is depressed ($b \rightarrow c$).

Since this depressed rate of combination ($b \rightarrow c$) is very slow, the amount of SO_3 given by point b indicates practically the minimum quantity of gypsum required to obtain a properly retarded cement. Since the amount of SO_3 combined as sulfoaluminate at the age of 10 min or more can be determined by Forsen's method, it may be possible to estimate the minimum gypsum requirement by applying Forsen's method to paste at age of 10 min or so.

To investigate this matter, penetration tests were carried out with cements that were ground in a laboratory mill from a commercial clinker and natural rock gypsum. Two ground samples were prepared. The first one contained gypsum equivalent to 0.70 percent SO_3; the second contained gypsum equivalent to 2.13 percent of SO_3. These two samples were blended to obtain several desired SO_3 contents. The results for the amount of SO_3 combined as sulfoaluminate at the age of 20 min by

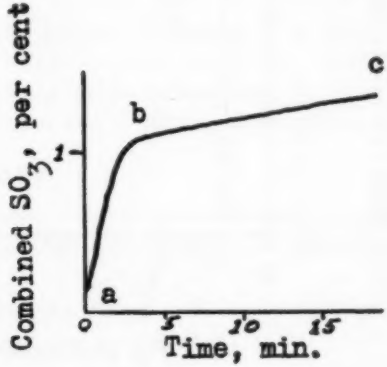

Fig. 4—Rate of combination of SO_3 as sulfoaluminate in paste

Forsen's method are given in Table 4, and the results of penetration tests are shown in Fig. 5.

In this case (Table 4), the amount of SO_3 combined as sulfoaluminate during the first 20 min was about 0.70 to 0.75 percent, whereas the minimum SO_3 requirement estimated from the penetration tests would be about 0.85 to 0.90 percent. The difference between these values of SO_3 was about 0.15 percent and such a difference was also found in experiments with other clinkers. Hence, by taking this difference into consideration the approximate minimum SO_3 requirement can be easily estimated.

Lerch[3] concluded that there was no difference between the effect of gypsum and that of plaster of Paris on the chemical reaction of cement. A clinker was ground with the natural rock gypsum in a laboratory mill. A portion of the cement was heated to 140 C for 3 hr. The amount of SO_3 combined as sulfoaluminate in pastes at the age of 20 min was determined for the original cement and the heated cement and was 0.69 and 0.91 percent, respectively. For a further study, two series of cements were ground in a laboratory mill to four degrees of fineness from a commercial clinker. The first series was ground with a natural gypsum and the second series was ground with a dehydrated gypsum which was prepared by heating the natural gypsum at 140 C for 4 hr. The

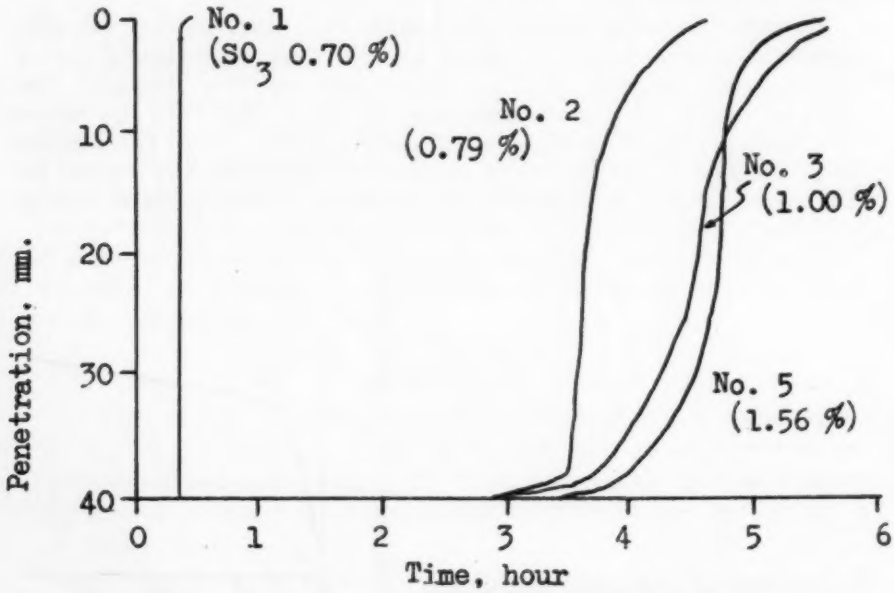

Fig. 5—Penetration test by Vicat needle

TABLE 5 — AMOUNT OF SO_3 COMBINED AS SULFOALUMINATE IN PASTE AT AGE OF 20 MIN BY FORSEN'S METHOD

Cements containing gypsum (SO_3 = 2.12 percent)		Cements containing dehydrated gypsum (SO_3 = 2.12 percent)	
Fineness by Blaine's method, sq cm per g	Combined SO_3 as sulfoaluminate, percent of total weight	Fineness by Blaine's method, sq cm per g	Combined SO_3 as sulfoaluminate, percent of total weight
2840	0.59	2790	0.60
3350	0.62	3350	0.69
3940	0.70	3950	0.81
4420	0.74	4220	0.86

amount of SO_3 combined as sulfoaluminate in pastes of these cements at the age of 20 min was determined by Forsen's method. The results are given in Table 5. These results show that there was a difference in the amount of SO_3 combined as sulfoaluminate between the two series. Hence some difference may exist between the effect of gypsum and that of dehydrated gypsum in the early hydration process.

In the case of cement that is slightly aerated and manifests false setting tendency, the combination of SO_3 does not take place within the first 2 or 3 min. The reason why the combination of SO_3 is delayed is not clear. As Hansen reported,[2] a reduction in the reactivity of aluminate as a result of aeration may be one of the reasons. As already seen in Fig. 3, the amount of SO_3 combined as sulfoaluminate in pastes that were prepared with cements aerated in moist air and containing a considerable amount of sulfoaluminate was decreased at first. These results perhaps indicate the formation of alkali carbonates which decompose the sulfoaluminate. Therefore, the formation of alkali carbonates by the aeration may also cause prevention of combination of SO_3. (The probable mechanism of false set caused by aeration was described in the discussion of Hansen's paper.[2])

Another consideration as to false setting properties may be possible. It is said, and perhaps is true, that the dehydrated gypsum formed during grinding causes false set through the crystallization of gypsum. Therefore, it appears that the tendency toward stiffening of paste depends on the amount of gypsum crystallized in paste, or in other words, on the amount of SO_3 which is not combined as sulfoaluminate in the first 5 min or so, if the gypsum in cement is completely dehydrated.

SUMMARY

To study the reaction rate of gypsum combining with calcium aluminate when cement contacts water, a new method has been developed using radioactive isotope sulfur-35.

Some investigations were undertaken by this newly developed procedure. The results are summarized briefly as follows:

1. When cement is mixed with water, a considerable amount of gypsum is combined in the form of calcium sulfoaluminate in the first few minutes and then the rate of combination becomes very slow.

2. In slightly aerated, false setting cement, the combination of gypsum occurs about 3 min later.

3. Also, when cement is aerated at high humidity, calcium sulfoaluminate is formed. In paste made from such a cement, the amount of SO_3 combined as sulfoaluminate decreases at first and then increases rapidly (Fig. 3).

ACKNOWLEDGMENT

We are indebted to Dr. Harold H. Steinour, who gave much valuable criticism on our investigation, especially suggesting an assumption from which Eq. (4) in the paper was derived.

REFERENCES

1. Forsen, L., "The Chemistry of Retarders and Accelerators," *Proceedings*, Symposium on the Chemistry of Cement, Stockholm, 1938, p. 298.

2. Hansen, W. C., "Aeration Cause of False Set in Portland Cement," *Proceedings*, ASTM, V. 58, 1958, p. 1044. Discussion by T. Manabe, *Proceedings*, ASTM, V. 58, 1958, p. 1055.

3. Lerch, W., "The Influence of Gypsum on the Hydration and Properties of Portland Cement Pastes," *Proceedings*, ASTM, V. 46, 1946, p. 1252.

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Title No. 56-39

Stresses in Deep Beams

By ELIHU GEER

Prestressed concrete I-beams with bearing blocks at their ends, when loaded to destruction failed in tension in the end block at its junction with the I-section. As this manner of failure is contrary to what known theories indicate, it was thought advantageous to investigate this matter more thoroughly. The purpose of this paper is to facilitate the design method of these end blocks by investigating stresses in deep beams, which are thought to be analogous to bearing blocks turned on their side.

The deep beam theory is in contrast with the flexural theory in that it is mainly one of distributing a localized force. One unusual feature of deep beams discovered in the course of the investigation is that the greatest tensile stress occurs *not* at the midspan but near the face of the support. Another interesting feature is that maximum stresses are a function of the magnitude of the load and not its location, e.g., the stresses caused by a load at the center of the beam are almost of the same magnitude as when the load is placed near the support. This study was limited to deep beams whose height was at least 0.5 of its clear span.

■ THE BUILDERS OF THE JOHN R. BRIDGE[1] near Detroit, Mich., found that every test beam for that project, when loaded to destruction, failed in tension in the end block at its junction with the I-section. Fig. 1 shows the situation. These beams were made of precast blocks, post-tensioned with strands anchored by bearing against end blocks. United States naval research investigators[2] found the same type of failure in monolithic concrete beams. Although the presence of tension in the ends of prestressed concrete beams had been known for a long time, these failures were of interest as they showed a marked departure from the pattern of stresses in theories and methods proposed by authorities in the field.

It was the purpose of this study to find a more suitable method for the design of end blocks for prestressed concrete beams of I-section. As the investigation revealed some significant and hitherto unreported phenomena in the subject of stresses in deep beams, this latter problem became a primary topic in the over-all study.

HISTORICAL BACKGROUND

Exact analysis of the stresses in a structural part bounded by several surfaces is impossible except in a few special cases. Since the introduction of the concept of elasticity in 1678 by Hooke,[3] there have been many contributions to the understanding of problems of this type. In 1888 Boussinesq[4] gave an expression for radial st. ess at any point in a semi-infinite elastic body subjected to a point load. Flamant[5] in 1892 expanded the solution to that for a linear load, and Mitchell[6] in 1900 furnished the solution in the case of load on a finite area.

When a body is limited in extent, as by the sides of a beam, ease of analysis is greatly reduced, and resort is usually made to approximate methods. In 1923 Bleich[7] gave a detailed statement of the use of trigonometric series in the solution of problems in plane stress. Tesar[8] in 1932 published the results of photoelastic analysis of the stresses in the ends of a beam of rectangular cross section with force applied endwise. This case is found in practice in prestressed concrete beams. In his book on prestressed concrete, Guyon[9] presents an elaborate and thorough discussion of stresses in end blocks. Tables, charts, and diagrams furnish the designer with a wealth of information for analysis and design. As in other works, the I-beam is treated as a special case of the prismatic beam. Guyon explains the origin and nature of tensile stress in the end of a prestressed concrete beam by means of stress trajectories or isostatics, thus furnishing an important key to the present study. Other investigators who have used this approach are Morice[10] and Pirson.[11]

DEEP BEAM PROBLEM

The departure of stresses in deep beams from straight-line distribution is well known. In 1932 Dischinger[12] used trigonometric series to determine the stresses in continuous deep beams. The Portland Cement Association prepared an expanded version[13] of Dischinger's paper and added solutions for simply supported spans. Chow, Conway, and Winter[14] used finite-difference equations to solve simple-span deep beam problems. In the discussion[15] of the latter paper, it was brought out that a considerable amount of error existed because of the coarseness of the net and because of the inherent rounding off of peak values.

Cheng and Pei[16] added much to the theory of deep beams by solving the case in which no displacement is permitted at the supports. Other

Fig. 1—Failure in end block of the John R. Bridge

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studies, including the present one, have assumed that the beams are free to slide on supports. Kaar[17] in 1957 reported on tests made on models of simply supported deep beams. Other papers on this subject have been published by Nylander and Holst;[18] Conway, Chow, and Morgan;[19] Bay;[20] Uhlman;[21] and others referred.

In view of St. Venant's principle, we should not attempt to apply conventional flexural theory, or to think in terms of bending moment, with reference to a beam in which no point is more than the amount of the height (or depth) of the beam from a concentrated force such as a load or a reaction. The present study is limited to beams having a height to clear span ratio of at least 0.5. End blocks frequently have proportions such that the ratio is 1.25 or greater.

A beam of typical proportions is shown in Fig. 2a. To bring out the origin of tensile stresses, a few stress trajectories are shown. At Point A, the trajectory shown by a solid line is concave inward. The stress along this line, being compressive, produces an outward resultant force as shown in Fig. 2b. There are many such trajectories in this vicinity, all producing outward resultant forces. The free left boundary can furnish no reactive force and consequently the cumulative effect is a tension in the direction of the dotted line. Incidentally, a study of Point B will indicate a possible source for tensile stress in the *top* of the beam. It is readily seen in a qualitative way that the action in a beam of these proportions is mainly one of the distribution of a localized force, as contrasted with the conventional flexural theory. Consequently the present study breaks away from the concepts of flexural stress except for a comparison of the magnitudes of the forces at the limits of the two theories.

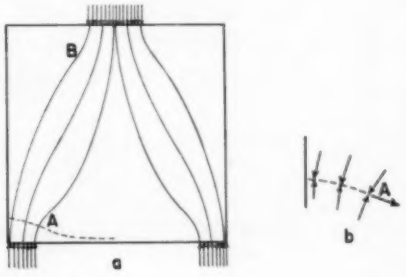


Fig. 2—Stress trajectories

PROCEDURE

Solutions for stresses in a variety of beams, as shown in Fig. 3, were made by finite-difference relationships between Airy stress functions on a square gridwork, somewhat as described in the appendix of Timoshenko and Goodier.[22] The grid size was one sixteenth of the length of the beam in all cases, so that one weakness, that of the coarse net, of the Chow, Conway, and Winter solutions[14] was overcome. The other weakness, that of the rounding off of peak values, was overcome by using five points instead of three for second derivatives. This was verified against the method of extrapolation from two networks given in books on numerical methods.[23]

VARIATION OF STRESSES RELATED TO LOCATION OF LOADS

Fig. 4, 5, and 6 show profiles of horizontal stress along the center line of span for various locations of load and for various ratios of depth to span. The following phenomena are of interest:

(a) Compressive stress in the top fiber increases as load is added near center of span but decreases when load is added in the outer thirds of the top, tension is caused in the top fiber.

(b) In the deeper beams, with load on the entire top, the maximum compressive stress occurs below midheight of beam.

(c) The familiar triple neutral axes are seen in the deeper beams with central loading. The upper two of these neutral axes disappear with shallower beam or with greater spread of load. The location of the lowest neutral axis is fairly constant for any given ratio of depth to length, but is proportionately lower in deeper beams. A rule of thumb is to assume that the tension exists in a depth equal to a third of the clear span.

(d) Total tension below the lowest neutral axis is approximately proportional to the total load on the span, especially for the deeper beams, when the loads are anywhere within the clear span. Total tension increases at a lesser rate when load is applied over the supports. Amounts are discussed later.

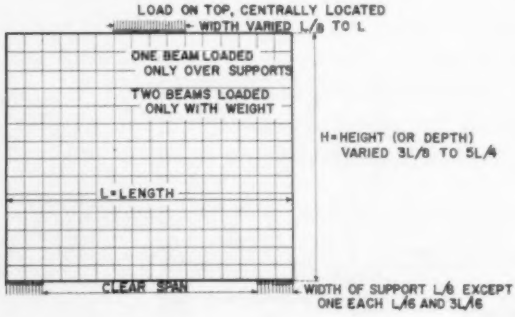

LOAD ON TOP, CENTRALLY LOCATED
WIDTH VARIED L/8 TO L
ONE BEAM LOADED ONLY OVER SUPPORTS
TWO BEAMS LOADED ONLY WITH WEIGHT
H=HEIGHT (OR DEPTH) VARIED 3L/8 TO 5L/4
L=LENGTH
CLEAR SPAN
WIDTH OF SUPPORT L/8 EXCEPT ONE EACH L/16 AND 3L/16

Fig. 3—Cases solved

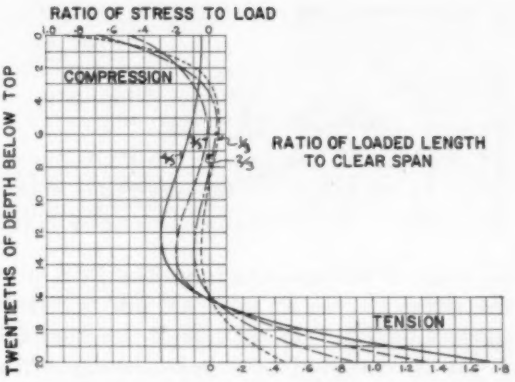


Fig. 4 — Variation in horizontal stress depth of beam = 5/3 clear span

Fig. 7 shows the variation along the span in horizontal stress in the lowest fibers for depth H equal to length L, with various portions of the length loaded. Startling departures from the conventional flexural theory appear therein. The greatest tensile stress is not at the midspan but is near to the face of support. Maximum tensile stress is practically independent of the location of the load, even to the extent that load placed only directly over the supports causes maximum stress of nearly the same amount as that caused by the same load placed on the central portion of the beam. There is of course in this case just as much tension in the top of the beam as in the bottom. The maximum intensity of stress in a deep beam depends on the amount of the reaction, not on the location of the load which caused it. It is somewhat disturbing to wonder if our shallower beams are similarly affected by local stresses at the reactions and loads.

The amount of error caused by assuming uniform distribution of reactive force was investigated. The ends were considered as continued into, and supported by, elastic bodies. It was found that the reactive stress was greater at faces of support, as could be expected, but that otherwise stresses were unaffected. Width of supports was taken as $L/8$

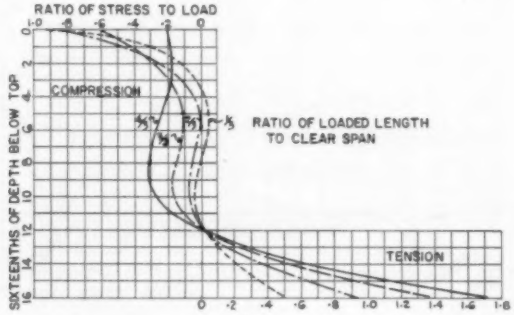


Fig. 5 — Variation in horizontal stress depth of beam = 4/3 clear span

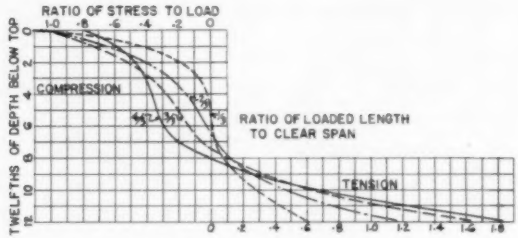


Fig. 6 — Variation in horizontal stress depth of beam = clear span

for most of the computations. The PCA pamphlet[13] stated in effect that the stress curves at midspan are nearly alike whether the width of support be 1/5, 1/10, or 1/20 of the span (center to center of supports). In the present study, one beam having a width of support equal to $L/16$ and another of $3L/16$ were solved. The total tensions at midspan compare closely with the beams having the same clear spans, depth, and loading but having supports $L/8$ in width. Maximum tensile stress is greater with narrower support.

Total tensile force

The designer of a concrete beam is interested in the total amount of the tensile force, and previous papers have tabulated this force. The PCA pamphlet used span center to center of supports. The Chow-Conway-Winter paper[14] used over-all length. Fig. 8 of the present study uses the ratio of height of beam to *clear span*, as it avoids introducing width of support and makes a definite break with the concept of bending moment.

The total tensile force is expressed as a fractional part of the total load. It will be noted that there are several curves depending on the spread of the load. In the discussion of the maximum tensile stress it was pointed out that its *intensity* was independent of the location or spread. Not so for the *total* tensile force. Just as in the conventional beam theory a point load creates more stress than the same load distributed over the length of the beam, so also in deep beams the total tensile force is greater if the force is over a shorter portion of the beam. Points for

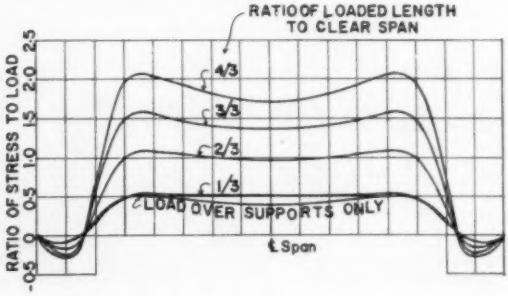


Fig. 7—Variation of stress along bottom

the curve marked zero, i.e., point load at midspan, were determined by extrapolation from the other curves.

It will be noted that width of beam has not entered into the computations. It is assumed that each unit width has the same unit loads and stresses.

To help decide on the ratio of height to span at which conventional theory should end and deep beam theory should begin, the total tension computed by the conventional theory is shown in dashed lines superimposed on Fig. 8.

Stresses due to weight of beam

Stresses due to weight of beam were solved for two ratios of depth to span. In other computations loads were applied at top of beam. Fig. 9 shows the dimensions of these beams. Fig. 10 shows the stresses at midspan and the stresses along the bottom. The stresses are practically the same as those due to the same loads applied at top of beam.

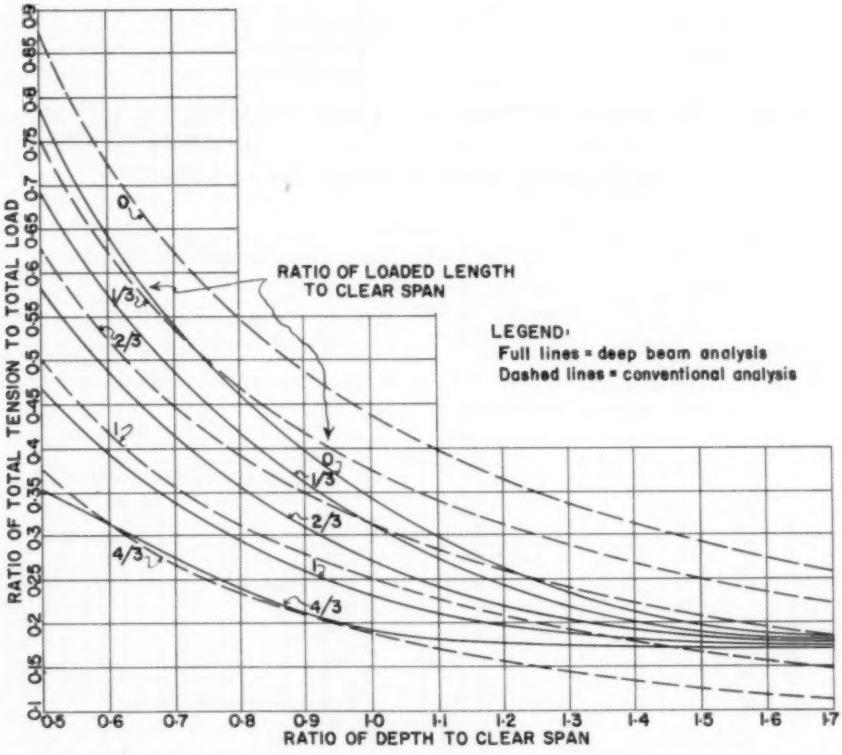

RATIO OF TOTAL TENSION TO TOTAL LOAD
0·1 0·15 0·2 0·25 0·3 0·35 0·4 0·45 0·5 0·55 0·6 0·65 0·7 0·75 0·8 0·85 0·9
RATIO OF LOADED LENGTH TO CLEAR SPAN
0
1/3
2/3
1
4/3
LEGEND:
Full lines = deep beam analysis
Dashed lines = conventional analysis
0·5 0·6 0·7 0·8 0·9 1·0 1·1 1·2 1·3 1·4 1·5 1·6 1·7
RATIO OF DEPTH TO CLEAR SPAN

Fig. 8—Tension in deep beams compared with corresponding amount by conventional beam theory

CONTINUOUS DEEP BEAMS

It is suggested that another careful look ought to be taken at continuous deep beams. Some of the most important papers have dealt with that case. Yet small variations in the conditions of end support can change the stress patterns completely. A small amount of shrinkage between unyielding ends can place every portion of the beam in tension. Movement of the ends due to external forces could create compressive horizontal stresses in every part of the beam. One possibility is that the sum of the horizontal forces acting against a section equals zero. In such cases investigators have found compression at the bottom face over the support but tensile stresses only a small distance higher.

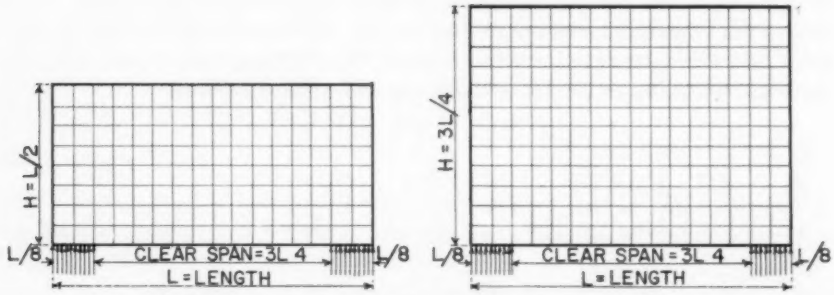


Fig. 9—Beams solved for stresses due to weight

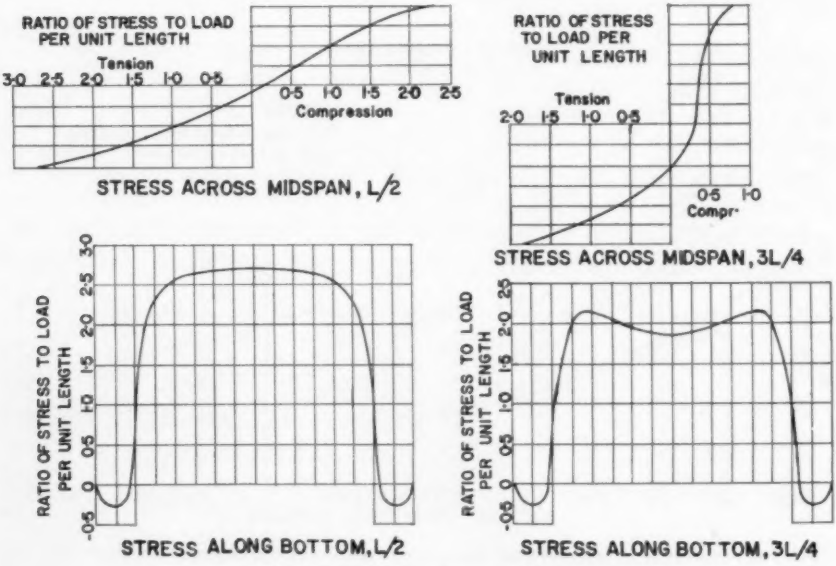
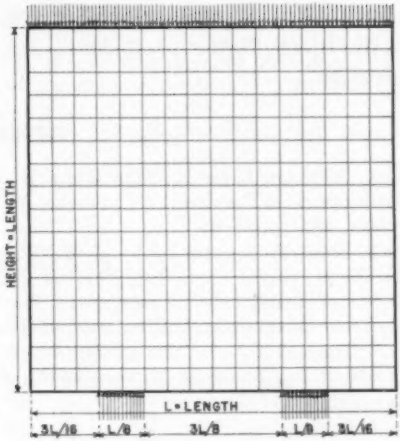

RATIO OF STRESS TO LOAD PER UNIT LENGTH
Tension
3·0 2·5 2·0 1·5 1·0 0·5
0·5 1·0 1·5 2·0 2·5
Compression
STRESS ACROSS MIDSPAN, L/2
RATIO OF STRESS TO LOAD PER UNIT LENGTH
Tension
2·0 1·5 1·0 0·5
0·5 1·0
Compr·
STRESS ACROSS MIDSPAN, 3L/4
RATIO OF STRESS TO LOAD PER UNIT LENGTH
3·0 2·5 2·0 1·5 1·0 0·5 0 -0·5
STRESS ALONG BOTTOM, L/2
RATIO OF STRESS TO LOAD PER UNIT LENGTH
2·5 2·0 1·5 1·0 0·5 0 -0·5
STRESS ALONG BOTTOM, 3L/4

Fig. 10—Stresses due to weight of beam

Fig. 11—Quasi continuous beam



If we have accepted the principle that stresses in a deep beam are a function of the spread of localized forces rather than due to bending moment, we should be prepared to expect to find tensile stress in the bottom only a short distance from face of support.

Fig. 11 shows a beam which is not continuous but which does have the same stress patterns at supports and midspan as does a continuous beam. Turned on its side, this beam becomes an end block of a prestressed concrete beam, with two bearing blocks. Fig. 12 shows the horizontal stresses on a vertical line through the center of support, on a vertical line at midspan, and along the bottom edge.

Examining the stresses over the support we can conclude that the beam needs reinforcement in the bottom over the support more than anywhere else in the beam. Incidentally this is the shape of stress profile used by Magnel, Guyon, Bruggeling, and others for the design of end

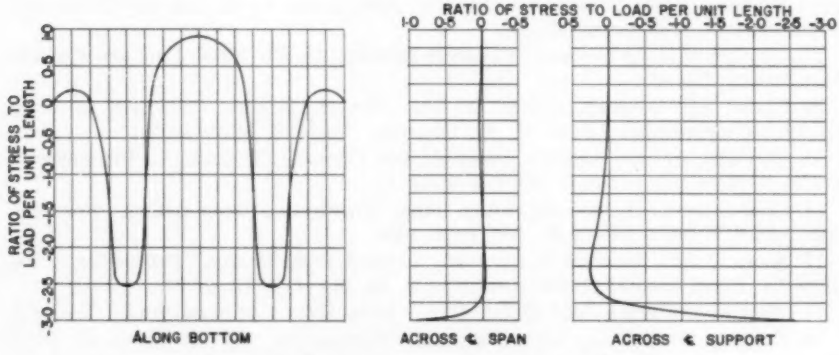

RATIO OF STRESS TO LOAD PER UNIT LENGTH
1·0 0·5 0 -0·5 -1·0 -1·5 -2·0 -2·5 -3·0
RATIO OF STRESS TO LOAD PER UNIT LENGTH
1·0 0·5 0 -0·5 -1·0 -1·5 -2·0 -2·5 -3·0
ALONG BOTTOM
ACROSS ℄ SPAN
ACROSS ℄ SUPPORT

Fig. 12—Stresses in quasi continuous beam

blocks. It is in good agreement when *there is uniform pressure* at one end of the block. However, it does *not* apply when the prestressed beam is of I-section.

REFERENCES

1. Johnson, Clair L., "The John R. Bridge," *Proceedings*, First United States Conference on Prestressed Concrete, Massachusetts Institute of Technology, Cambridge, 1951, p. 61-64.

2. Steel, R. K., and Libby, J. R., "Preliminary Tests of Prestressed Concrete at the U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, California," *Proceedings*, First United States Conference on Prestressed Concrete, Massachusetts Institute of Technology, Cambridge, 1951, pp. 231-240.

3. Hooke, Robert, "De potentiâ restitutivâ," as reported by Isaac Todhunter and Karl Pearson in *A History of the Theory of Elasticity*, I, p. 5. The discovery was made in 1660.

4. Boussinesq, "Memoir," as given in some detail by Todhunter and Pearson, *op. cit.*, II, Part II, pp. 240-47. Boussinesq also discussed deformations caused by loads on circular areas in his *Applications des Potentials*. . . .

5. Flamant, *Comptes rendus des séances de l'Académie des Sciences*, V. 114, 1892, p. 328; *Annales des Ponts et Chaussees*, Memoirs, Part 2, 1893, p. 228.

6. Mitchell, *Proceedings*, London Mathematical Society, V. 32, 1900, p. 35.

7. Bleich, Friedrich, "Der Gerade Stab Mit Rechteckquerschnitt Als Ebenes Problem," *Der Bauingenieur*, 1923, pp. 255-259, pp. 304-307, pp. 327-331.

8. Tesăr, V., "Experimental Determination of the Stresses at the Ends of Prismatic Members with Imperfect Joint," *Publications*, International Association for Bridge and Structural Engineering, I, 1932, pp. 497-506.

9. Guyon, Yves, *Béton Précontraint*, Editions Eyrolles, Paris, 1951, pp. 169-255, and Annexe I.

10. Morice, P. B., "Plotting Trajectories of Principal Stress in Plane Stress Problems," *Engineering* (London), V. 174, No. 4517, Aug. 22, 1952, pp. 252, 253.

11. Pirson, E., "Considérations sur les abouts de poutres en béton précontraint. Expose dune méthode de calcul basée sur l'entude des trajectories des efforts," *Annales des Travaux Publics de Belgique* (Brussels), V. 106, No. 3, June, 1952, pp. 401-436.

12. Dischinger, Franz, "Beitrag zur Theorie der Halscheib und des wandartigen Balkens," *Publications*, International Association for Bridge and Structural Engineering, I, 1932, pp. 69-93.

13. "Design of Deep Girders," *Concrete Information* No. ST66, Portland Cement Association, Chicago.

14. Chow, Li; Conway, Henry D.; and Winter, George, "Stresses in Deep Beams," *Proceedings*, ASCE, V. 78, Separate No. 127, May, 1952.

15. Guzmán, Arturo; Luisoni, Cesar J.; and Conwell, William A., *Proceedings*, ASCE, V. 79, Separate No. D-127, Feb. 1953.

16. Cheng, David H., and Pei, Ming-Lung, "Continuous Deep Beams," *Proceedings*, ASCE, V. 80, Separate No. 450, June, 1954.

17. Kaar, P. H., "Stresses in Centrally Loaded Deep Beams," *Proceedings*, Society for Experimental Stress Analysis, V. 15, No. 1, 1957, p. 77.

18. Nylander, Henrik, and Holst, Hans, *Deep Beams of Reinforced Concrete*.

19. Conway, Henry D., Chow, Li, and Morgan, G. W., "Analysis of Deep Beams," *Journal of Applied Mechanics*, V. 18, No. 2, June, 1951, pp. 163-172.

20. Bay, H., *Uber der Spannungszustand in hohen Tragern und die Bewehrung von Eisenbetontragwanden*, p. 64.

21. Uhlman, H. L., "The Theory of Girder Walls with Special Reference to Reinforced Concrete Design," *The Structural Engineer* (London), V. 30, No. 8, Aug. 1952, p. 172.

22. Timoshenko, S., and Goodier, J. N., *Theory of Elasticity*, McGraw-Hill Book Co., Inc., New York, 2nd Edition, 1951.

23. Salvadori, Mario G., and Baron, Melvin L., *Numerical Methods in Engineering*, Prentice-Hall, New York, 1952.

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CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Construction

Reinforced concrete props as mining supports (in Polish)

W. Milkowski, W. Perkowski, F. Rubel, *Prace Glownego Instytutu Gornictwas* (Stalingrad), No. 207, 1958, 12 pp.

Polish Technical Abstracts No. 1 (33) 1959

Describes reinforced concrete columns, 4⅝ in. in diameter, that are used to support rock roofs in mining shafts. These columns are investigated with regard to the effect different types of heads (used for capping) have on the ultimate bearing strength.

Development of the design of storage tanks in Hungary (in Hungarian)

B. Gnadig, Gy. Markus, and J. Thoma, *Vizugyi Kozlemenyek*, No. 2, 1958, pp. 133-165

Hungarian Technical Abstracts V. 11, No. 1, 1959

Describes the development after World War II of field storage tank construction. Some of the more noteworthy tanks designed by the authors are described. One is a lentiform reinforced concrete tank of 3000 cu m capacity. In the construction of this tank, as compared to older types, the use of framework and scaffolding can be almost entirely eliminated and the amount of concrete and steel is considerably reduced.

Comparative analyses on the economy of approximately 120 tanks of various sizes show that with tanks exceeding 300 cu m capacity the joint application of prefabrication and stressing resulted in an average saving of 43 percent concrete, 45 percent steel, and 80 percent timber. Different stressing methods have been introduced: (a) stressing by the vertical oblique displacement of the wires wound horizontally around a cylindrical tank, (b) stressing by the oblique displacement of the wires wound around a conical tank, (c) stressing by the radial displacement of the wires wound around a cylindrical tank.

Winter concrete placement completes ICBM base on time

Contractors and Engineers, V. 56, No. 10, Oct. 1959, pp. 34-36, 41-42

The $12 million contract to build the first squadron launching facilities for intercontinental ballistic missiles near Cheyenne, Wyo., was scheduled for completion in 390 calendar days. There was no alternative but to carry on the work through the winter at maximum speed. Actually, there were few days during the entire winter when concrete was not being placed and other work was also in progress. The contractor made suitable provisions to fight the cold. Work, carried on simultaneously on three of the launchers, was protected with enclosures of plywood, canvas, and clear plastic film. While the basic reinforced concrete frame of each launcher was being built, almost the entire structure was enclosed in a housing. Then, after the concrete had been placed and cured, portions of the

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enclosures were removed and re-used on other structures. Openings in the concrete walls were then closed with temporary doors or with coverings of plastic film so that work could continue inside. Heat inside the enclosures was provided by steam boilers and unit heaters, and oil burning heaters. 28,000 cu yds of concrete was produced on the site. Water for the mix was heated to 135 F by a steam boiler, but the dry ingredients were not heated. Two cranes took the concrete from the transit mixers and hoisted it to the forms. The workmen opened the canvas roof covers just enough to permit concrete to be placed and vibrated. Then the covers were replaced, and a temperature of at least 50 F was maintained inside the enclosure for a 7-day curing period. Exposed surfaces were sprayed with curing compound. A crew of about 300 men worked through the winter to keep the project on schedule and as weather became more favorable, the crew was augmented until it reached a peak of about 600 men.

New method for concreting shafts in South African gold mines (Ein neuartiges Verfahren zum Betonieren von Schachten aus dem sudafrikanishen Goldbergbau)

ADOLF MEYER and M. H. THOMPSON, *Bau-Maschine und-Technik* (Wiesbaden), V. 6, No. 8, Aug. 1959, pp. 293-302

Mine shafts lined with concrete or reinforced concrete offer many advantages both as regards costs and performance, provided they are properly designed and executed. The shaft described will be sunk to a depth of 1950 m and will have a diameter of 8.38 m. Before sinking operations began, cement grout was injected into the earth down to 720 m. The shaft sinking operations were not interrupted by other concrete work. The concrete was conveyed from the concrete plant to the point of use by pipes. Up to 60 cu m of concrete can be handled per hour by two pipes having an internal diameter of 152 mm. The sinking platform has three decks and also carries the mechanical lashing equipment. The steel forms consist of a 80 cm high ring and a 3.80 m high liner plate. Stripping is generally done after 8 hr. Three sections (13.80 m) were usually executed in 2 days. Details are given on soil stratification, structural and mechanical sections, equipment, and construction procedure.

Construction Techniques

Torsion in concrete piles during driving

BEN C. GERWICK, JR., *Journal*, Prestressed Concrete Institute, V. 4, No. 1, June 1959, pp. 58-63

Reports on the cracking and resulting failure of concrete piles during their installation and the ensuing investigation that was conducted to determine the cause or causes. The investigation indicated that the prime factor causing cracking was the torsional reaction of the driving head on the pile, acting alone or in combination with the tensile rebound stresses. This twisting of the piles by the driving head was eliminated by having all pile heads rounded by a manual chipping process. This together with careful driving procedure eliminated all subsequent failures.

Construction of apartment houses with cellular, sandless, large blocks in the far north (in Russian)

E. I. KRICHEVSKAIA, *Bulletin Stroitelnoi Tekhniki* (Moscow), V. 15, No. 11, Nov. 1958, pp. 36-39

Reviewed by HERBERT E. KESE

In the Soviet Union, regions above the arctic circle have no sand, so crushed stone of basaltic rock (which is in abundance there) is used as raw material for manufacturing large wall blocks to build houses. The article describes the building practices for this method of construction.

Area for a standard apartment house is 960 sq m and is normally four stories

in height. The material for both exterior and interior building walls is large precast concrete blocks. The wall thicknesses are 64, 51, and 40 cm, correspondingly, and the size of the blocks varies between a maximum size of 178.5 x 137.5 x 64 cm, used in exterior walls, to a minimum size of 88.5 x 51 x 40 cm used for building interior walls. Weight of a single large block 1.5 to 2.0 tons. The building blocks are mass-produced at a concrete mixing plant, and transported to the building site by trucks.

Proportions of concrete mix for the manufacturing of blocks is 1:10 or 1:12, i.e., 1 part portland cement to 10 or 12 parts of crushed stone, no sand or fine aggregates are used. The graduation of the aggregate is between 10 and 20 mm while the water-cement ratio is given as 0.45. Addition of 4 percent gypsum by weight is recommended to increase the strength of blocks. After the concrete is placed into forms, they are automatically conveyed into a chamber for steam curing, a process of 18 hr duration. Next step is the removal of forms and storage handling. The article states that this method of construction is found to be the most economical for that particular region.

Ganged forming helped complete bridge on time

Roads and Streets, V. 102, No. 8, Aug. 1959, pp. 82-83

Describes the use of pre-assembled forms for constructing a large number of girders for a bridge in Gary, Ind. These prefabricated forms saved on labor and time.

The changing face of concrete

Concrete Construction, V. 4, No. 8, Aug. 59, pp. 1-6

Describes a variety of techniques available that are used to alter the appearance of concrete surfaces from a dull flat finish to something more decorative. This is done in two basic ways, they can be applied during the forming stage before the concrete has hardened, or they can be applied to the finished concrete surface. Some of the surface effects achieved are wood-textured, striated, exposed aggregate, textured, rough or dimpled, and patterned.

Use of liquefied bituminous binders for curing concrete (in Polish)

S. Wojdanowicz, G. Wojdanowicz, *Drogownictvo*, No. 7, 1958, pp. 168-171
Polish Technical Abstracts No. 1 (33), 1959

Reports on the use of liquefied bituminous binders for curing freshly laid concrete. Study indicates that increasing the volatility of the solvent gives a more watertight curing film. This process reduces the cost of curing by 24 to 32 percent.

CRSI recommended practice for placing reinforcing bars

Concrete Reinforcing Steel Institute, Chicago, 1959, 287 pp., $3

This is a handy pocket-size manual which contains much valuable information and many practical hints on reinforcing bar placement not available in textbooks. Although originally written as a guide for reinforcing bar setters, it was expanded to include information for detailers, engineers, architects, contractors, and field inspectors. The manual summarizes the best of the accepted current practices in the placement of reinforcing bars, welded wire fabric, and their supports in reinforced concrete structures. It should be a valuable aid to teachers and students of architecture and engineering.

The main section includes hints on unloading and storing reinforcing bars, preassembly, handling of bars, and placing plans and specifications. Bar supports and spacers, and the placing of bar supports in various types of construction are covered thoroughly. Also included are sections on field welding, tie wire, tolerances in placement, and welded wire fabric. Specifications and

detailed instructions are given for placing bars in footings, columns, foundations, walls, floors, and beams. There is a section on inspection of reinforcing bars in place. In addition to describing various types of reinforced concrete construction, there is a general section describing individual structural members.

Nuclear plant demands new construction idea

ANTHONY N. MAVROUDIS, *Contractor and Engineers*, V. 56, No. 9, Sept. 1959, pp. 22-24, 26, 27

Reports on the construction techniques used in the building of a nuclear power station. The biggest job at the plant was the construction of the 160 ft diameter steel containment sphere and its external concrete shielding wall and dome.

The 181-ft-diameter shielding wall, which is 90 ft high, was widened from 5½ ft to 7½ ft for the last 20 ft of its height. This circular wall was prestressed by having 275 miles of wire wrapped around its top. Top was prestressed to provide inward thrust to counteract lateral thrust, caused by the weight of 24 precast, reinforced concrete dome ribs, which form the roof over the steel sphere. The specifics of prestressing are discussed.

Dams

Handbook on multiple buttress dams (Sbornik prac o clenenych priehradach)

Slovenske vydavatelstvo technickej literatury (Bratislava), 1955, 938 pp., $10
Reviewed by J. J. POLIVKA

This voluminous handbook is outstanding as to the thoroughness of general description, design, and construction of these special types of dams which are used economically all over the world. Authors of 25 chapters are recognized specialists: Chapter 1 by L. Hobst: structural layout and arrangement, spillways, flood gates. Chapter 2 by L. Hobst, A. Korvas, and J. Laudat: describes various type dams, especially those built in Russia and Czechoslovakia. Chapter 3 by K. Lossmann: foundations of multiple-buttress dams under various soil conditions. Chapter 4 by Lossman: construction procedure, study of geological conditions, properties of water, diversion of streams, installation of construction equipment, mixing and placing of concrete, supervision during construction, and technical and economical evaluation of the dam after completion and during the following years. Chapter 5 by A. Korvas: organization and mechanical installations on construction site, transportation of equipment, and building materials, etc., thoroughly discussed in 28 examples. Chapter 6 by L. Hobst: use of prefabricated (precast) structural units and their assembly. Chapter 7 by A. Pavlik: presents methods and procedures used in manufacturing precast, prestressed, structural members; new method of manufacturing precast members under lower atmospheric pressure based on research performed in Russia, properties of vacuum aggregates, lifting and transportation of precast members, and methods of their assembly. Chapter 8 by J. Stork: technology of concrete, properties of materials and concrete for this type of dam. Chapter 9 by A. Pavlik: vibration of concrete, its theory and effects on properties and quality of concrete, bond of reinforcement, adhesion between old and new layers, vibration under water and necessary equipment, preferable methods of vibration, and test results. Chapter 10 by L. Hobst: scaffolds, forms, and movable shoring for plain and hollow buttresses. For interconnecting slabs and shells, a description is given of the bracings, materials of forms (wood, plywood, steel, combination of wood and steel, wire mesh and cloth), effects of forms on quality of concrete, forms with absorbing surface, forms in reinforced concrete. Chapter 11 by L. Hobst: design of expansion joints in foundation slabs, buttresses, retaining struc-

ture, and sealing materials. Chapter 12 by L. Hobst: rigid reinforcement, its advantages and applications. Chapter 13 by L. Mejzlik: structural design, while considering factors of safety and economy, most economical spacing of buttresses, and shape of retaining shells (advantages of the Noetzli type). Chapter 14 by L. Jejzlik: loads to be considered in design, such as; static and dynamic water pressure, effect of waves and ice, gravity load, uplift, effects of temperature changes on shrinkage of concrete, stresses in foundations, earthquake stresses, combination of maximum stresses with stresses due to mechanical prestressing of the dam. Chapter 15 by K. Konrad: temperature changes, conductivity of air, water and materials, radiation, reciprocity of radiation of two solid bodies, thermal connection and transmission, their coefficients and analytical expression. Chapter 16 by L. Mejzlik: determination of stability and safety for various inclinations of buttresses. Analysis and its criteria are shown in numerical example. Chapter 17 by P. Peter: effects of underground water pressure on seepage and their prevention, spillways, scour pipes, hydraulic disturbances, erosion, effects of freezing, various methods of stabilization, injection of cement and other materials, and chemical soil solidification examples. Chapter 18 by L. Mejzik: analysis of restrained arched shell with example. Chapter 19 by J. Brilla: two-hinged arch with three curvatures. Chapter 20 by Z. Cermak and J. Brilla: analysis of concrete arches restrained by buttresses and foundation slab (for all possible loadings). Chapter 21 by Z. Cermak: analysis of the crown structure. Chapter 22 by J. Babuska, Fr. Vycichlo, Fr. Kroupa, and K. Rektorys: analysis of triangular dams on elastic foundations. Chapter 23 by J. Brilla: analysis of buttresses. Chapter 24 by L. Mejzik: analysis and design of foundations. Chapter 25 by Z. Cermak: experimental stress analysis.

Design

Intrinsic theory of torsion (in Spanish)

GUNTER LUMER, *Facultad de Ingenieria y Agrimensura* (Montevideo), No. 16, Sept. 1958, pp. 15-22
Reviewed by EMILIO ROSENBLUETH

Derivation of equations pertaining to the elementary theory of torsion of prismatic beams using abbreviated tensor notation.

Contribution to the analysis of plates with locally varying slab thickness (Ein Beitrag zur Berechnung ortlich dickerer Platten)

GUNTER HOELAND, *Beton und Stahlbetonbau* (Berlin), V. 54, No. 3, Mar. 1959, pp. 65-69
Reviewed by RUDOLPH SZILARD

Whereas the analysis of isotropic and orthotropic slabs is facilitated by the use of extensive tables, slabs with locally varying slab thickness (like slabs with haunches) are not treated. The investigation of such problems can be attacked by the solution of differential equations covering different thickness zones or by iteration, which has been used by the author. The first approximation, for instance, in computing influence fields can be taken from tables assuming uniform slab thickness, this value will be corrected by an additional value. The method is illustrated by a numerical example.

Helical stairs (in Spanish)

F. CASSINELLO, *Informes de la Construccion* (Madrid), V. 10, No. 100, Apr. 1958, 14 pp.
APPLIED MECHANICS REVIEWS
May 1959 (Sciammarella)

Purpose of the paper is to provide the engineer with an easy tool for the analysis and design of helical stairs. Author points out the difficulties of a rigorous analysis, and develops the problem on the basis of the simplifications of the beam theory. The differential equations of equilibrium for a beam element are derived, using stress resultants across normal sections. The integration of six simultaneous linear differential equations leads to six integration constants, depending on the

boundary conditions of the beam. These constants are given by a system of six linear equations that are obtained considering the beam as a redundant structure. In deriving the coefficients of the equations, bending and torsion deformations are considered, and shear and normal deformations are neglected. Solution is carried out for a beam double-clamped at both ends. The final matrix can be split into two matrices, one of the second order, for symmetric loading; one of fourth order, for antisymmetric loading. Paper is complemented with a numerical example.

Effects of torsion in tall buildings (in Spanish)

J. RICALDONI, *Facultad de Ingenieria y Agrimensura, (Montevideo)*, No. 16, Sept. 1959, pp. 1-14

Reviewed by EMILIO ROSENBLUETH

Review of present elementary methods for computing shear distribution among frames of a given floor in a building subjected to eccentric lateral loads. Author considers with some detail the case in which drift of each frame is assumed proportional to the horizontal shear taken by that frame in the floor under consideration. He gives an indication of procedures to be followed when flexural deformations of a shear wall are not negligible. Also included are standard formulas for computing shear stiffness of frames and various types of bracing. Paper is clearly written and gives a good, even if brief, bibliography.

Practical calculation of reinforced concrete (Calcolo pratico del cemento armato)

GIORGIO NEUMANN, 2nd Edition, Edizioni Cremonese, Rome, 1958, 292 pp., 3200 L.

Reviewed by J. J. POLIVKA

Presents thorough information on concrete as structural material with emphasis on recent improvements and applications (shells, domes, and other space structures). Part I treats characteristic properties of the materials, nature, proportioning of mixes, their control, inspection before and after concreting, and testing. Part II refers to design and calculations, including details and specifications, of various structural types and members such as foundations, columns and pilasters, floors, beams, frames, arches, roofs, shells, and domes. Simplified methods of calculation are presented, accompanied by numerical examples and tables. Various admixtures and their properties are discussed and full information presented on lightweight concrete and its aggregates, precast and prepacked concrete, and various construction methods. Special attention is given to floors with hollow block introduced in Italy by the author half century ago. Also the monolithic combination of three materials, concrete, steel, and glass ("glass-concrete") and its advantages is described.

Introduction to the dynamics of framed structures

GROVER L. ROGERS, John Wiley & Sons, Inc., New York, 1959, 355 pp.

The author presents in a logical manner an introductory exposition on the theory of structural dynamics as it pertains to the special problems of beams and rigid frames. He deals with the particular problems involved in the design of bridges and buildings and pitches the material at a mathematical level.

He discusses the combined systems using a matrix formulation that makes possible a straightforward attack on systems having multidegree of freedom, with or without damping, and subjected to arbitrary disturbances. The book treats the problem of distributed mass systems, using the theory of orthogonal functions as the mathematical tool and extends the moment balancing procedures of structural statics to the subject of structural dynamics. Author relates the formulation of all problems to the direct applications of Newton's laws of equilibrium.

After discussion of dynamic disturbances, the book goes into structures

with one degree of freedom, two degrees, and multidegrees, then develops vibrations, inelastic beams, continuous beams, rigid frames, and floor systems. Final chapter discusses certain energy methods including the equations of Lagrange.

The reader should be conversant with the statics of frame structures and have an interest in the application of mathematical methods to all structure's problems.

Some uses of matrix methods in structural analysis

J. L. Meek, *Civil Engineering Transactions*, Institution of Engineers, Australia (Sydney), V. CE 1, No. 2, Sept. 1959, pp. 81-89
Author's Summary

The first section outlines the use of the Maxwell-Mohr superposition equations, and stresses that these equations give a rapid solution to structural problems with techniques now available. Suitable techniques, based on matrix algebra, are given for solving these equations using high-speed desk calculators. The second section is a brief introduction to the "force method" of analyzing structures using matrix methods suitable for programming on automatic digital computers.

Elastic stability computation of unsupported reinforced concrete frames (Knickberenchnung freistehender Stahlbetonrahmen)

Alfred Habel, *Beton und Stahlbetonbau* (Berlin), V. 54, No. 2, Feb. 1959, pp. 25-31
Reviewed by Rudolph Szilard

The article treats the practical buckling problem of the most common reinforced concrete frames based on the idealized materials following Hooke's law. According to the assumptions, loads are acting on the joints only. Buckling formulas are derived for the columns and beams of simple frames with hinged and fixed columns, three-legged frames with the similar boundary conditions as above. Variation of the moment of inertia is also considered. Buckling of multistory building frames is treated, also. Due to the complexity of the problem certain approximations are inevitable. For cases not handled in the article merely an "educated guess" is recommended (for unsupported length mostly the distance to the moment zero points) which is according to the author, sufficient for most practical cases.

Concrete silos

T. J. S. Mallagh, *Transactions*, Institution of Civil Engineers of Ireland (Dublin), V. 84, June 1957-May 1958, pp. 123-146

Discusses pressures in deep silos, size and shape of bin, hoppers, methods of bin discharge, and methods of construction. Examples of blockwork silos are given. Drawings show plans and sections for different types of silos. One chart gives the calculation of factor of safety for loads in accordance with Baker's weighting method for three types of grain silos: blockwork with central discharge at moderate rates, concrete hoppers; in situ concrete, with central discharge at moderate rates, concrete hoppers; and in situ concrete with offset discharge at rapid rates, and/or steel hoppers.

Industrial building details

Duane F. Roycraft, F. W. Dodge Corp., New York, 2nd Edition, 1959, 356 pp., $12.75

This is the first handbook of its type presenting over 1500 architectural details for contemporary industrial buildings. These details cover every part of the industrial building and include structural details, walls, windows, louvers, roofs, parapets, monitors, floors, ventilators, stack curbs, expansion joints, flashing, canopies, doors, door frames, stairs, ladders, platforms, hatch covers, manholes, storage and transportation facilities, and many more. These details have all been taken or adapted from actual construction drawings and many include dimensions that apply to the specific building, or buildings, in which they were incorporated. A highly useful key to symbols and abbreviations is given and a complete index enables the designer to locate any detail in a moment. The introductory

comments in each chapter are intended to aid proper selection of details by explaining features or operating problems that may not be obvious from an inspection of the drawings themselves. This should be a valuable reference for architects, engineers, designers, draftsmen, plant engineers, engineering departments, and industrial building contractors.

Proposed design for reinforced concrete structures

P. J. Carroll, *Progressive Architecture*, V. 50, No. 6, June 1959, p. 165

It is proposed that the usual factor of safety on the dead load be reduced in the design of reinforced concrete structures. Although there would be higher stresses, but not excessive, in the steel and concrete, the author points out six primary advantages that would result.

Plastic analysis of structures

P. G. Hodge Jr., McGraw-Hill Book Co., New York, 1959, 364 pp., $10.50
Applied Mechanics Reviews
July 1959

Book presents the techniques used in the analysis of structures beyond the elastic limit and the conditions under which a plastic analysis is appropriate, safe, and economical. Structures treated include beams, frames, shells, plates, and slabs under static and variable loadings, including dynamic effects. Two basic theorems are emphasized which enable lower and upper bounds on the loading capacity to be determined by independent consideration of static and kinematic conditions respectively. Book has two parts: Part I treats in detail the application of plastic methods to bending of beams and frames, and also deals with elastic-plastic deformations, variable and repeated loading, and direct procedure of design. Part II is primarily concerned with combined stresses in beams, circular plates, and cylindrical shells. A brief introduction to some of the problems encountered in the dynamic loading is included. At the end of each chapter problems with examples are thoroughly discussed. Extensive bibliographies and references are given.

The planning of concrete tunnels

T. Harosy, *A Magyar Tudomanyos Akademia Muszaki Tudomanyok Osztalyanak Kozlemenyei*, V. 23, No. 1-2, 1958, pp. 39-62
Hungarian Technical Abstracts
V. 11, No. 2, 1959

The strength conditions of rock surrounding a tunnel possess a far greater degree of uncertainty than the play of forces in the tunnel supports. Therefore the conventional procedure used in designing (that of first establishing rock pressure and subsequently conforming the dimensions of the tunnel supports) has been abandoned for the reverse procedure. A simple method for determining the stresses in the supports and their deformation in tunnels of circular cross section is introduced by means of a fundamental load pattern. This is verified by a numerical example. The load pattern is based on the maximum roof pressure and the minimum side pressure for whose theoretical determination some easy to manage and well-known formulas approximating actual conditions are mentioned.

Analysis of cylindrical shell roofs with longitudinal prestress by the substitute prismatic shell method (in German)

G. Schmausser, *Bautechnik* (Berlin-Wilmersdorf), V. 34, No. 2, Feb. 1957, pp. 44-49
Applied Mechanics Reviews
May 1959 (White)

Method consists in replacing the curved shell by a series of flat plates; it has been used previously for roofs without prestressing. The prestressing wires are assumed to be either straight or with parabolic curvature in plane normal to surface. The force they exert on the roof is equivalent to two (for straight wires) or three (parabolic wires) external loads on each roof plate: (1) a longitudinal centric force at each end; (2) a couple at each end; (3) a uniform loading normal to the

plate. Following cases are treated: long shells ($L/B > 2$); intermediate shells ($1 < L/B < 2$); single longitudinal span; single or multiple lateral spans; unsymmetrical loading and shape laterally; symmetrical loading and shape longitudinally; smooth load variation: constant thickness of each plate (not necessarily shell); prestress wires in shell and/or edge beams anchored at ends of roof.

Calculation for the carrying capacity of concrete compounds of rectangular section taking into consideration the prevention of crack appearance (in Russian)

G. P. Orlenko, *Trudi Novacherkas, Politekhn In-ta*, No. 33-37, 1956, pp. 171-190
Applied Mechanics Reviews
June 1959

Formulas are derived for determining the load carrying capacity of concrete beams subjected to deflection, axial tension, and eccentric tensile and compressive forces. Formulas are derived so as to limit crack width.

Materials

New concrete joint sealant testing machine

R. J. Schutz and A. Van Hauter, *ASTM Bulletin*, No. 241, Oct. 1959, pp. 42-44

Describes a machine to test the various qualities of joint sealant materials. Some of the tests already carried out on the machine are described briefly.

Fly ash concrete after 7.5 years

William L. Chilcote, *Baltimore Engineer*, V. 33, No. 8, Nov. 1958, pp. 4-6
Highway Research Abstracts
May 1959

In October 1958, a test section of highway was placed in Baltimore using fly ash in one section and air-entrained concrete in another. Six bags of air-entrained cement were used in both cases.

At the time the highway sections were placed, 6 x 12 in. test cylinders were made so as to be able to make tests on the relative strengths of the two mixes for a period of 1 year. Except for the first 90-day period, the compressive strength of the fly ash concrete has been consistently greater than that of the plain concrete.

Compressive tests at about 3½, 5, 6½, and 7½ years have subsequently been made by taking 6 x 9 in. cores from the existing roadway pavement. The strength of the fly ash mixture has continued to be consistently greater than that of the regular cement mixture. After 7½ years, the compressive strength of the fly ash mixture was found to be 6355 psi, while normal concrete mixture tested at 5605 psi.

Effect of calcium chloride on the properties of portland-blast furnace slag cement (in Japanese)

Setsuzo Suzuki, *Semento Gijutsu Nenpo*, V. 12, 1958, pp. 164-71
Ceramic Abstracts
Sept. 1959 (Suzukawa)

Flash or abnormally slow set due to the presence of an insufficient amount of gypsum is changed to normal set by the addition of a proper amount of $CaCl_2$. The addition of 1 to 3 percent, by weight, of $CaCl_2$ increases the strength of portland-blast furnace slag cement at early ages but has practically no influence on strength at later ages. The flow of mortar and the slump of concrete increase with the addition of a proper amount of $CaCl_2$, owing to increase in the entrained air content.

Effect of calcium chloride on the formation of calcium sulfoaluminate during hydration of cement (in Japanese)

Nobue Fukuda and Nobuo Adachi, *Semento Gijutsu Nenpo*, V. 12, 1958, pp. 67-71
Ceramic Abstracts
Sept. 1959 (Suzukawa)

The content of residual uncombined SO_3 in neat cement paste (water/cement = 0.30) within 5 min after mixing with H_2O decreases with the addition of 2 percent by weight $CaCl_2$ to the cement. In cement containing a large amount of SO_3, the formation of calcium chloroaluminate seems to occur after 5 hr. In the presence of $CaCl_2$, the effect of

a low curing temperature, 3 C, in decreasing the uncombined SO_3 content is also shown.

Structural concretes — methods of controlling proportions by iteration (in Polish)

W. KUCZYNSKI, *Budownictwo i Architectura* (Warsaw), 1956, pp. 220
Reviewed by J. J. POLIVKA

Describes the fundamentals and nature of structural concrete such as: proportioning of mixes, composition of paste, hardening processes, curing, frost resistance, shrinkage and swelling, inspection and testing of cement and aggregates, and other properties. References are given test results in Poland and other countries.

Pavements

Ready-mixed concrete for use in highway pavements

GEORGE R. BATHE, *Concrete Construction*, V. 4, No: 8, Aug. 59, pp. 15-26

Ready-mixed concrete producers will find this article a helpful introduction into the possibilities and problems related to serving highway construction jobs with ready-mixed concrete.

The manner of meeting rigid specifications, what they consist of, method of persuading contractor to use product, and the cost and profits issue are all thoroughly discussed.

Pavement design for commercial jet aircraft

F. M. MELLINGER, R. G. AHLVIN and P. F. CARLTON, *Proceedings*, ASCE, V. 85, May 1959, pp. 29-43
HIGHWAY RESEARCH ABSTRACTS
July 1959

Pavement requirements are given in the form of design charts for both rigid and flexible pavements. The design curves cover a range of loadings for the Boeing 707, Douglas DC-8 and Convair 880 commercial jet aircraft. The basis and application of the pavement design curves are fully explained. The strengthening of existing rigid pavements by overlays is covered by references. There is also a brief discussion of structural requirements for jet aircraft pavement as compared to present requirements for the more critical propellor-driven aircraft pavement. The requirements for airport pavements given in this paper are limited to the pavement structure.

Monolithic anchors control end movement of continuous pavement

STAFFORD E. THORNTON, *Concrete Construction*, V. 4, No. 8, Aug. 59, pp 11-13

Describes a 300 ft long, 5 ft wide, continuous pavement that employs no expansion or contraction joints, constructed for the purpose of investigating end movement of slabs. Because there are no expansion joints, the end of a continuous slab may move as much as 4 in., thereby causing difficulties joining new pavement with existing structures. A solution to this problem was the employment of a series of keyways or lugs at the end of the pavement, to act as anchors and prevent movement.

Construction of continuous reinforced concrete pavement at Dundee

M. H. WALLER, *Surveyor* (London), V. 117, 1958, pp. 839-842
ROAD ABSTRACTS
May 1959

Describes design and construction of the section of the Perth-Aberdeen-Inverness Trunk Road. Concrete was laid in two vibrated layers. During the placing of the first 5⅓-in. deep layer, longitudinal tie-bars were inserted through the formwork. Reinforcement was delivered in sheets measuring 17 x 6½ ft, weighing 10.76 lb per sq yd, in accordance with British Standard 1221 Part A. It was placed 2⅔ in. from the top of the slab, on the lower concrete layer, with a 9-in. center overlap (16 in. at transverse joints). Two crack surveys after completion of concreting disclosed only minor cracks. After the survey a 2-in. carpet of two-coat asphalt was laid over the concrete slab.

Precast Concrete

Precast T-frames speed library erection

Engineering News-Record, V. 162, No. 25, June 25, 1959, pp. 34-36

Describes the use of precast reinforced T-frames, precast beams, and precast roof panels for the construction of a college library at Grinell, Iowa. The method of analysis, 28-day compressive strength, curing, and erection are briefly outlined.

Prefab apartment houses in Kallhall, Sweden (in Swedish)

LENNART HULTEBERGER

Construction of prefab apartment houses (in Swedish)

AKE HOLMBERG

Production and erection of room-sized elements (in Swedish)

NILS HANSSON

Aspects of finish in prefab construction (in Swedish)

STURE JACOBSSON
Vag Och Vatten Byggaren (Stockholm), No. 2, Mar. 20, 1959

Reviewed by CARL H. GUSTAFSON

In these four articles are presented all of the factors and solutions of the problems in the construction of 360 prefab apartments. The problems discussed are the training of personnel, material handling, formwork "en masse," erection schedule, construction details, and wall finishes.

Here's a roundup of European floor-and-roof systems

Concrete Products, V. 62, No. 10, Oct. 1959, pp. 41-47

Countries of Europe in rebuilding the war-torn cities have been more inclined to experiment with precast concrete than have producers in the United States. In Holland, precast concrete floor and roof design has received considerable attention and, for the size of the country, has either developed or borrowed an amazing number of systems. Holland uses five different types of floor and roof designs: precast beams; precast slabs; beam and slab combinations; joists and block combinations; and, finally, a wide variety of plain block systems. This is Part 1 of the article dealing with the first three types. Discusses and pictures seven kinds of precast beam floor and roof systems, the three top beam and slab systems, and three precast and three lightweight slab systems. Also tabulates the technical information for the systems concerning height, weight, and maximum span for different capacities.

Prestressed Concrete

Application of prestressing in building with prefabricated concrete members (Die Anwendung der Vorspannung beim Bauen mit Betonfertigteilen)

LEVICKI, ERNST, and DRESDEN, *Betonstein Jahrbuch* 1959, Bundesverband der Betonsteinindustrie E. V. (Bauverlag GMBH, Berlin-Wiesbaden), pp. 78-152

Reviewed by J. F. LEPPMANN

A world wide survey of structural applications of prefabricated prestressed concrete elements and their field connections. Both mass produced (mostly standardized) and individually fabricated members are covered, including floor and roof constructions, crane girders, bridges, silos, piles and sheet pilings, tunnel and subway linings, track ties, and transmission poles. Problems of economy, fabrication, transport, and erection are touched upon.

Some important prestressed silos and reservoirs in Scandinavia

CHR. OSTENFELD and E. KALHAUGE, *Journal*, Prestressed Concrete Institute, V. 4, No. 1, June 1959, pp. 18-34

Discusses European prestressed concrete tanks and silos and their essentially different line of development and use as compared to those made in the United States.

The Europeans use silos and tanks for the storage of many different materials such as: cement, sugar, coal,

mineral oils, cement slurry, and in some cases boiling water. In contrast, the United States uses tanks mainly for the storage of water and sometimes oil.

In regard to structural differences the Scandinavians, who are most active in this field, normally design their tanks as monolithic structures, therefore without sliding joints between wall and bottom slab. Their prestressing cables are placed inside the concrete and not outside as in the United States. Other interesting features of construction and design are discussed and should provide valuable information to those doing similar work.

Partial prestressing and possibilities for its practical application

P. W. Abeles, *Journal*, Prestressed Concrete Institute, V. 4, No. 1, June 1959, pp. 35-51

Introduces and explains in general terms the difference between full, and partial prestressing. The difference being that partial prestressing allows for some tensile stresses when under working load whereas in full prestressing no tensile stresses are allowed.

The advantage of using partial prestressing is, reduction of cross section and a decrease in prestressing force which results in saving anchorages and labor cost with post-tensioning.

Several examples are given, both in this country and England, of buildings and structures employing the partial prestressing system.

Building code requirements for prestressed concrete

Subcommittee on Prestressed Concrete, Structural Engineers Association of Northern California, *Journal*, Prestressed Concrete Institute, V. 4, No. 1, June 1959, pp 72-78

Article, in effect, is the proposed draft of a building code on prestressed concrete that is intended to be included in the Uniform Building Code. These tentative recommendations are based on the report of ACI-ASCE Joint Committee 323. A few of the sections covered: definitions of terms such as, losses, forces, transfer, etc.; modulus of elasticity; ultimate strength theory; allowable stresses; prestress losses; web reinforcement; ultimate flexural strength; and pretensioned construction.

Properties of Concrete

Micrometer method for measuring the volume stabilities and weight changes of thin layers of portland cement paste during carbonation and drying

K. M. Alexander and J. Wardlaw, *Australian Journal of Applied Science* (Melbourne), V. 10, No. 2, June 1959, pp. 214-217
Author's Summary

A micrometer method is described for measuring changes in length of thin layers of hydrated portland cement paste stored continuously in controlled atmospheres. Length and weight changes are measured without disturbing or interrupting the curing process. The procedure can be used for specimens with cross sections less than 0.01 sq in., and is particularly suitable for studying the effects of carbonation and drying.

Effectiveness of silicones as water repellents when applied to concrete

F. C. Lanning, *Transactions*, Kansas Academy of Sciences (Manhattan), No. 61, 1958, pp. 334-344. *Chemical Abstracts*, V. 53, No. 9, May 10, 1959, p. 8579
Highway Research Abstracts July 1959

To determine the water repellency of various silicones when applied to concrete block, units made with expanded shale aggregate and with heavy aggregate, and with standard and air-entrained portland cement, were tested. Three types of silicones were used: solutions of methyl, ethyl, or other silicone resins dissolved in volatile solvents, such as toluene or mineral spirits; an aqueous solution of NaMe siliconate; and a MeOH-silicone solution. Test procedures are described, and the results are evaluated. One conclusion of special significance is that silicones are not effective for waterproofing very

porous material, such as block made with expanded shale. Several of the silicone preparations were found to be effective in treating less porous block, in which sand and gravel were used as aggregate.

Aggressive action of water on cement and steel

B. Visintin, *Giornale del Genio Civile* (Rome), V. 96, No. 5, 1958, pp. 282-286. *Building Science Abstracts*, V. 31, No. 10, Oct. 1958, p. 292

Highway Research Abstracts
July 1959

To determine the liability of concrete and steel water-pipes to corrosion, the soil in which the pipe is to be laid should examined from the geological, chemical, and electrochemical viewpoints, and in addition an accurate chemical analysis should be made of the water to determine the following characteristics: its acidity expressed in pH, its specific electrical conductivity at 18 C, its total alkalinity, its total and permanent hardness, and its free carbon dioxide content. Standard methods of determining these are outlined and discussed briefly.

Control of concrete strength by the ultrasonic method (in Russian)

O. E. Pflaumer, *Avtomatika Dorogi*, V. 22, No. 1, 1959, pp. 11-12

Road Abstracts
May 1959

Describes the ultrasonic control method used during the construction of a reinforced concrete bridge across the Moscow river. To check the variables involved in concrete manufacture which affect the accuracy of ultrasonic readings, control test cubes were prepared at the factory from the same material under identical conditions and subjected to ultrasonic and compression tests. A comparison of the results obtained showed that concrete of different strengths can sometimes give the same ultrasonic reading. Concrete strength was determined from the average of nine readings of each of the precast sections tested in place. Further control tests were made on cubes taken from the completed bridge components. Examples are given of the type of information obtained and of the calibrated curves which relate ultrasonic data to those obtained in compression tests. The IKL-5 ultrasonic instrument was found to be suitable for dimensions of 4½ to 6½ ft and to have an accuracy of 0.5 μ. The results of the ultrasonic and compression tests did not differ by more than 6 to 10 percent.

Experimental study of the elastic-plastic properties of concrete, obtained by a new technological process (with wet cement grouting) (in Russian)

A. E. Sheikin and N. S. Baskakov, *Mosk. Inta Inzhenerny*, No. 80/1, 1955, pp. 400-426

Applied Mechanics Reviews
May 1959 (Grigoryan)

Comparative data are given in regard to the strength and the elastic-plastic properties, with repeated compression and creep, of concrete prisms 10 x 10 x 30 cm in the course of 3, 7, and 28 days. The samples were prepared in the usual way and with a wet cement grouting. The compressive strength of the samples with the ground cement addition exceeded the strength of the ordinary samples by 25 percent in the course of 3 days. With the lapse of time this superiority became less and after 28 days was only 7 percent greater. In testing for repeated compression the load was increased in stages and ranged from 0.2-0.8 of the compressive strength of the above sample. The samples tested for creep were loaded in a 7-day increase of the axial compression force, equal to half the disintegration force in this loading stage. Similarly, the settlement deformations were measured for the unloaded samples. The results obtained in the tests showed that the use of the wet ground cement addition in no way exercised any appreciable influence on the elastic or residual deformations for comparable applications of stress to the regular concrete. In the earlier growth-stage of the concrete (3 days) the wet

ground cement addition lowers somewhat the elastic-plastic properties of the concrete. The wet ground cement addition reduces the creep deformation of the concrete in the initial period of the load (30-45 days); in a 3-day period the load action by 27 percent; in a 7-day, by 15 percent. In the later periods the difference between the relative creep deformations of the samples with the ground cement addition and without decreased. Toward the 180th day the creep deformation for samples having wet ground cement addition was 14 percent greater than the sample deformation without the wet ground cement addition, which the authors explain by the increase of the degree of stress in the samples with wet ground cement addition.

Shrinkage characteristics of concrete (in Russian)

S. V. Alexandrovsky, *Beton i Zhelezobeton* (Moscow), No. 4, Apr. 1959, pp. 169-174
Reviewed by D. Watstein

The author states that when prismatic specimens used in shrinkage studies are permitted to dry on all sides, the deformations resulting from the three-dimensional moisture gradient interfere with the "pure" shrinkage and obscure its true value. The paper presents the results of careful measurements of shrinkage and moisture changes in 5 x 5 x 11-cm specimens which were permitted to dry only through the ends. For such specimens the drying of the concrete is not accompanied by shrinkage until a certain "critical" moisture content is attained. The "critical" values for a given cement depends on the w/c ratio, the age at which drying begins and the cement factor.

The relationship between the shrinkage and the moisture loss of a given concrete is essentially linear. The slope of this graph which the author terms the coefficient of linear shrinkage is reported to be constant for a given cement and is independent of the water-cement ratio, proportions of the concrete and its age. For an ordinary concrete this coefficient is given by 0.03 (mm/mm)/(g/g).

Diffusion of water in mortar, especially the effects of capping methods and of the direction of water permeation (in Japanese)

Yutaka Kuriyama and Yuko Shigekura, *Semento Gijutsu Nenpo*, V. 12, 1958, pp. 128-32
Ceramic Abstracts Sept. 1959 (Suzukawa)

Mortar specimens of 15 x 15 x 4 cm were cured in air at 20 ± 2 C for 28 to 32 days, dried at 70 ± 2 C for about 70 hr until the weight remained constant, and then tested under a 30-cm head of water. Water permeability is not affected by capping with a trowel, but it does vary with the face of specimens; permeability from the upper face is greater than that from the lower face.

Discontinuous shrinkage of hardened portland cement pastes and mortars subjected to continuous drying

K. M. Alexander and J. Wardlaw, *Australian Journal of Applied Science* (Melbourne), V. 10, No. 2, June 1959, pp. 201-213
Author's Summary

Experiments are described which show that when water is lost continuously from hydrated portland cement pastes and mortars the resulting shrinkage takes place in two stages. Total water in these pastes and mortars should therefore be considered as Stage I evaporable water, Stage II evaporable water and nonevaporable water. If the period of water curing which precedes the drying operation is increased, the proportions of nonevaporable water and Stage II evaporable water increase at the expense of Stage I evaporable water. On increasing the water-cement ratio the Stage II evaporable water increases only slightly, almost all the extra water lost on subsequent drying being released during Stage I. Following curing periods of 7 to 180 days, shrinkage during Stage I of water loss is about twice that which takes place during Stage II. Within the above

range of curing times, increasing the period of water storage increases both Stage I and Stage II drying shrinkage. The increased drying shrinkage caused by raising the water-cement ratio appears to be due entirely to increase in the Stage I shrinkage.

Influence of the direction of loading on the strength of concrete test cubes

A. M. Neville, *ASTM Bulletin*, No. 239, July, 1959, pp. 63-65

Discusses the question, does the direction of the application of the load, relative to the direction of casting, give different results for similar test specimens.

One report states the strength of mortar cubes tested in the direction of casting was between 0.87 and 0.91 of the strength of similar cubes tested at right angles to the direction of casting.

States, after testing 16 batches of 12 cubes each, that if the test specimens are approximately equal, then "the direction in which the load is applied relative to the direction of casting is not expected to influence their strength."

Deformation of concrete in compression (in Russian)

K. E. Thal, *Investigations on the Strength, Plasticity, and Creep of Building Materials*, Moscow, 1955, pp. 202-207

Applied Mechanics Reviews
July 1959

Reviews the results of axial compression tests on prisms of reinforced and nonreinforced concrete. The concrete had a crushing strength of 105 kg per sq cm (prism test) while the reinforcement was of cold-drawn wire with a nominal yield point in compression of 5000 kg per sq cm.

The values of the mean limiting strain were, for the nonreinforced prisms, $\approx 2 \times 10^{-3}$, and for the reinforced prisms, twice that, ie.., $\approx 4 \times 10^{-3}$. Stress curves for the concrete and the reinforcement are given; the stresses in the reinforced prisms reached a maximum deformation of $\approx 1.2 \times 10^{-3}$ and a load of 80 percent of the maximum. With further increase of the load, the stress in the concrete decreased, because of redistribution of the load to the reinforcement, while the deformation value abruptly increased.

Owing to technical difficulties, it was impossible to observe directly during the test the phenomenon of appreciable growth of the plastic deformations in compressed concrete, accompanied by decreasing compression stresses.

Corrosion of concrete sewers

South African Council for Scientific and Industrial Research, Pretoria, Series DR 12, 1959, 236 pp. £1

This book is divided ino three parts plus two appendices on test methods for determination of chemicals and gases in sewage. Part I deals with field investigation of sewer corrosion, introduction on the mechanism of sewer corrosion and occurrence and effects of hydrogen sulphide and sulphuric acid. Discusses results of studies and surveys in South African sewer systems with field experiments to test effects of aeration, cleaning of sewers, addition of chemicals and modifying design of systems. Part II deals with the microbiological aspects of sewer corrosion describing bacteriological methods and types of surveys and studies of inhibitors. Part III deals with concrete technology in relation to sewer corrosion with methods for testing effect of acid on portland cement concrete and methods for improving resistance of concrete to acid and sulphate attack. Concludes that pressure steam curing of concrete will impart greater resistance to corrosion but the effect is relatively small and the cost involved is high. Suggests that the most effective remedial measures would be the substitution of a calcareous aggregate for the siliceous aggregate normally used and the use of a limestone aggregate in preference if a suitable one is available. Aims for as rich a mix with as low a water-cement

ratio as is practical and economical. Also recommends the use of cements of higher sulphate resistance than normally found with portland cements, for application such as mortars and plasters exposed to sewer gases.

Structural Research

Deformation measurements for structural testing

LADISLAV B. KRIZ and OTTO A. KURVITS, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 1, No. 3, Sept. 1959, pp. 35-41

AUTHORS' SUMMARY

Instrumentation and methods commonly used in the Portland Cement Association Structural Laboratory are described. Deflections are measured by micrometer dial gages, precision leveling, or precision triangulation. Installation of SR-4 strain gages on concrete and reinforcement is illustrated, and the use of electronic instruments for strain measurement is outlined.

Experimental verification of shell roofs by means of models (Comprobacion experimental de cubiertas laminares, por medio de modelos reducidos)

C. BENITO, *Informes de la construccion* (Madrid), No. 111, May 1959, No. 112, June-July, 1959, Item 466-3

Interesting and detailed discussion on the use of models to verify design of shell structures. After a general introduction to problems related to shells, author considers scale models, their size, material, and loading. The final sections deal with plaster models, more nearly equal in behavior to concrete.

Influence of creep in a composite (steel-concrete) beam defined by $n(\zeta)=E_s/E_c\xi$ (in German)

ST., BLASZKOWIAK, *Bautechnik* (Berlin), V. 35, No. 3, Mar. 1958, pp. 96-100

Reviewed by J. J. POLIVKA

Author uses Swiss specifications for determining creep effect in composite bridge structures by introducing combined modulus of elasticity of concrete (*Schweizerische Bauzeltung* (Zurich), July 28, 1956) and compares the results with those obtained by his method. Author and his students applied both methods to an example of a composite bridge structure calculated in Sattler's book " Theorie der Verbundkonstruktionen" (Theory of composite structures). The results of both methods are tabulated and the conclusion is that the author's method, which is based on standard procedure used for such structures without considering creep effects, complies with Swiss regulations. Author's modifications are thoroughly discussed and illustrated, and demonstrate the considerable simplification of the analysis.

Elastic stability of reinforcing bars subject to short and long time loading (Knicken von Stahlbetonstaben unter Kurz- und Langzeitbelastung)

KURT GAEDE, *Issue* No. 129, Deutscher Ausschuss für Stahlbeton, Berlin, 1958 (Publisher Wilhelm Ernst and Sohn), 80 pp., 24 DM

Reviewed by RUDOLPH SZILARD

This important scientific publication of DAS (German Concrete Institute) consists of two parts: the first part covers the extensive test reports, the second the theoretical investigations. Test specimens having 10 x 15.4 cm cross section reinforced by 1 percent longitudinal steel and 2 mm diameter spirals with 10 cm pitch were subjected to test. The quality of concrete and reinforcing steel has been varied, coupled with variation of the L/r ratio (102 to 123). The specimens were exposed to short time loadings as well as to long time loadings with eccentricities $e_1 =$ 2 cm and $e_2 = 5$ cm. All the specimens in the testing machine had statically determinate pin supports at both ends. In addition to the magnitude of the loads, the lateral displacements and concrete strains were measured.

With large eccentricities, tension failures governed the ultimate load carrying capacities of the specimens, whereas with small eccentricities the compression failure of the concrete

governed. Important finding of the test is that under long time loading the ultimate load carrying capacity is markedly reduced by the creep of the concrete. The influence of the creep is a function (besides the well known other variables as properties of the concrete, atmospheric condition, etc.) of the e/d ratio, where e represents the eccentricity and d the diameter of the reinforcing bars, respectively. With larger eccentricity this influence tends to be negligible.

It is expected that the new German reinforced concrete bridge and building codes (DIN) is going to reflect the findings of the research since deviation up to 25 percent on the unsafe side can result from the use of the present codes. Since the above described research was carried out for statically determinate columns, further research is required for statically indeterminate compression members like columns of multistory building frames.

Tests on 15-year-old, 54-ft-long pretensioned beams

G. D. Base and H. E. Lewis, *Journal*, Prestressed Concrete Institute, V. 4, No. 1, June 1958, pp. 64-71

Describes tests conducted on two pretensioned I-beams that had been stored, unloaded, for 15 years. The main object of the tests was to determine the loss of prestress that had occurred in 15 years of storage (17 percent), but tensile strength, ultimate loads, and deflections of beams were also measured.

Fundamental photoelastic studies on skrinkage stresses in massive structure

Toshikazu Kawamoto, *Proceedings*, Eighth Japan National Congress for Applied Mechanics, 1958, Science Council of Japan (Tokyo), Mar. 1959, pp. 225-230

The fundamental experiment and the approximate solution were studied for the states of stress caused by shrinkage of new concrete which had been placed on old concrete or on foundation rock. The author made a fundamental experiment on the shrinkage stresses by an application of photoelectric methods, explained that it is possible to find the states of stress in the mass of structure subjected to shrinkage, and performed the approximate calculation to estimate the stresses in concrete blocks in the foundation, to compare with the experimental results. This work treated only the case in which the concrete block had a rectangular shape and elastic properties the same as the foundation material.

General

Some observations on the movement of buildings on expansive soils in Vereeniging and Odendaalsrus

L. E. Collins, Symposium on Expansive Clays, Part I, *Transactions*, South African Institution of Civil Engineers (Johannesburg), V. 7, No. 9, Sept. 1957, pp. 273-285
Reviewed by D. G. Norman

Results of observations of 37 houses founded on expansive soils, including one conventional, 24 on underreamed piles of different depths, and 12 of reinforced brick-work. Maximum, average, and differential movements are given, also data on soil-swelling at various depths.

Engineer Eugene Freyssinet and his work (in Rumanian)

E. Prager, *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 9, No. 12, 1957, pp. 706-718
Reviewed by J. J. Polivka

Biography describing one of the leading engineers and inventors in the field of reinforced and prestressed concrete. His activity as a prominent engineer, architect, and contractor is discussed in connection with a great number of his structures which are generally well known.

Fire resistance ratings

National Board of Fire Underwriters, Revised Edition, Apr. 1959, 62 pp.

This edition is a revision of the January, 1957, edition. It contains a number of tables of types of construction which provide fire resistance ratings

required by building codes or regulations. Data covered include beam, girder, and truss protections; ceiling constructions; column protections; floor and ceiling constructions; roof constructions; and walls and partitions. The tables are set up in terms of minimum requirements (type, details of construction, and thickness of materials) for the specified fire resistance and in such form as to fit the requirements for 4-hr fire resistance given in building codes.

Work of the European Committee for Concrete

A. L. L. Baker, *The Structural Engineer* (London), V. 36, No. 1, Jan. 1958, pp. 10-20
Reviewed by C. P. Siess

Describes organization and work of Comité Européen du Béton and reviews concisely but adequately the work which has been done and the points on which agreement has been reached. This interim report is concerned chiefly with methods of computing the ultimate strength of members subjected to moment, shear, and axial compression individually or in combination: definitions of failure; and factors of safety. Recommendations for future testing are summarized.

Rain resistance of walls for housing in South Africa

S. J. P. Joubert, *Research Report* DR 11, National Building Research Institute, Pretoria, 1957
Reviewed by D. G. Norman

Application to thin walls of National Building Research Institute standard method of simulating rain.

European research on cement and concrete

Inge Lyse, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 1, No. 3, Sept. 1959, pp. 4-13
Author's Summary

Presents the general problem of research on cement and concrete in Europe today and discusses some of the more important subjects which are being investigated. A summary of research activities of 14 West European countries is given in the survey. Finally a brief review is given of the different research laboratories and the most important investigations in progress and in each one of the 14 countries.

Concrete and prestressed concrete engineering in the USSR

T. Y. Lin, Boris Bresler, David P. Billington, Ben C. Gerwick, Jr., James D. Piper, and Walter H. Price, Portland Cement Association, 1959, 86 pp.

A comprehensive report of a tour of concrete construction projects and prestressing and precasting plants in Moscow and Leningrad. Like previous reports by individual members of the delegation, this complete report casts much light on the progress of Russian concrete technology.

Elsevier's dictionary of building construction

C. J. Van Mansum, D. Van Nostrand Co., Inc., New York, 1959, 472 pp., $15.75

A four-language (English-American, Dutch, French, and German), 8500-item compilation of terms in the building field used by contractors, architects, tradesmen, and office personnel. Tabular arrangement is based on the English language with corresponding terms in the other languages arranged horizontally. Word finding starting with Dutch, French, or German is facilitated by alphabetical lists in those languages keyed to the numbers assigned to the English equivalents.

Russian - Polish engineering and building dictionary (in Polish and Russian)

Warsaw, 1956, 583 pp.
Polish Technical Abstracts No. 3 (31), 1958

Comprises roughly 29,000 Russian terms and their Polish equivalents, together with definitions in Polish. Definitions are for words currently being used, in architecture, civil engineering, building materials and products, economy, organization of building enterprises, and for pure and applied sciences in relation to the building trade.

ACI NEWS LETTER

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1959 AWARDS ANNOUNCED

The 1959 ACI award winners have been announced by the Board of Direction following approval of the Awards Committee report. The Henry C. Turner Award, the Alfred E. Lindau Award, the Henry L. Kennedy Award, and the Wason Medals for research and the most meritorious paper will be presented this year; 11 individuals in all will receive awards.

The awards will be presented at the 56th annual convention to be held in New York, March 14-17, at the Commodore Hotel.

Henry C. Turner Award to Harrison F. Gonnerman

In recognition of numerous and outstanding contributions to concrete technology through research and administration.

This award, established in 1927 by the late Mr. Turner, ACI past president, is presented for notable achievement in, or service to, the concrete industry. The medal is of gold and is accompanied by an engraved certificate that recounts the medal's purpose.

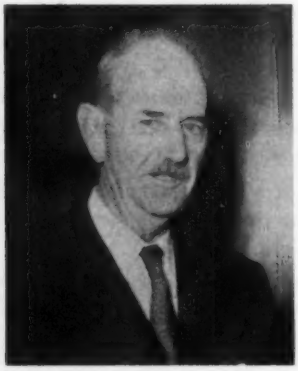
Eminent in the field of research, Mr. Gonnerman has twice received the ACI Wason Medal for noteworthy research, in 1929 and 1944. He received the Sanford E. Thompson award from ASTM in 1952 and the ASTM Award of Merit in 1956.

Mr. Gonnerman attended the University of Illinois, Urbana, receiving his BS degree in 1908 and MS in 1913. He served in the Engineering Experiment Station, University of Illinois, and

Harrison F. Gonnerman

later was instructor in theoretical and applied mechanics. In 1952 he retired from the Portland Cement Association, Chicago, after 30 years service on the PCA research and development staff, and established an office in Oak Park, Ill., as consulting engineer and research consultant.

Mr. Gonnerman joined ACI in 1918 and has participated in almost every phase of Institute endeavor. In 1951 he was awarded honorary membership in the Institute. He served as president in 1947, having previously filled the post of vice-president and membership on the Board of Direction. He is a Quarter-Century Member-Getter.



Arthur R. Lord

A prolific writer, Mr. Gonnerman has written countless technical papers, many of which have been published in the JOURNAL.

Mr. Gonnerman has also contributed his time and efforts to the activities of the American Society of Civil Engineers, American Association for the Advancement of Science, Western Society of Engineers and the Highway Research Board.

Alfred E. Lindau Award to Arthur R. Lord

As the *originator of reinforced concrete building tests: a pioneer authority in the development of flat slab construction.*

This award was founded in 1947 by the Concrete Reinforcing Steel Institute to honor the memory of Alfred E. Lindau, past president of the American Concrete Institute, who died at Pearl Harbor, Dec. 14, 1944, while serving as chief civilian engineering assistant to the Officer in Charge, Pacific Naval Air Bases. The token of the award is a bronze plaque bearing a bas-relief portrait of Mr. Lindau. It is given only for outstanding contributions to reinforced concrete design practice and is not restricted to ACI members.

Arthur R. Lord is a pioneer in Institute affairs and in concrete design practices. He has made many important contributions to the progress of concrete during his lifetime as an engineer.

Mr. Lord graduated from the University of Maine in 1907. During the next few years he taught at the University of Illinois. His curiosity to know more about the design and control of concrete led to the development of many "firsts" for the industry. He helped develop strain gages for use on concrete structures; he instigated and carried through the first actual loading test of a building as a basis for stress analysis. He was active in analysis of flat slab construction and pioneered in a variety of structural designs. During the 1920's

he was the world authority on flat slab design.

Becoming an ACI member in 1916, Mr. Lord plunged into committee work and contributed generously to the JOURNAL pages in the years that followed. His "Handbook of Reinforced Concrete Design" (1928) was widely circulated and added prestige to ACI Building Codes. The inception of the present JOURNAL is largely due to his efforts. In 1934 he was installed as ACI president after having served as vice-president and as a member of the Board of Direction. In 1927 he was awarded the Wason Medal for the most meritorious paper of the year, "Notes on Concrete—Wacker Drive, Chicago." He was made an Honorary Member of the Institute in 1957.

Henry L. Kennedy Award to Harry C. Delzell

In recognition of his outstanding contributions to the advancement of the objectives of the American Concrete Institute.

This award, established in 1958, in memory of the late Henry L. Kennedy, past president of ACI and chairman of the Building Committee, is awarded for outstanding technical or administrative service to the Institute. The bases for selection of awardees are outstanding activity or service that has enhanced the Institute's prestige, marked leadership in technical, administrative, or special committee work, or other distinguished service to the Institute. The award is in the form of a framed scroll.

Harry C. Delzell, managing director, Concrete Reinforcing Steel Institute, Chicago, has been a dynamic force in educational efforts in the reinforced concrete design field. His efforts have had a widespread influence throughout the engineering profession and have lead to better use of concrete in construction. Over a period of 20 years, Mr. Delzell has shown a sustained interest in the welfare of the American

Harry C. Delzell

Concrete Institute and has been generous of his time and talents in the furtherance of ACI objectives.

Mr. Delzell has worked on a number of ACI technical and administrative committees and served two terms on the Board of Direction. In 1946 he was instrumental in arranging for the release to ACI Committee 315, Detailing Reinforced Concrete Structures, of the "Proposed Manual of Standard Practice for Detailing Reinforced Concrete Structures," on which much time, effort, and expense had been devoted by the Engineering Practices Committee of CRSI. This Manual later became a Standard of the Institute with revisions in 1948, 1951, and 1957 to reflect changes in the ACI Building Code and improvements in industry practices.

Mr. Delzell coordinated the work of AASHO, CRSI, and ACI Committees to expedite the adoption by ACI in 1953 of the "Manual of Standard Practice for Detailing Reinforced Concrete Highway Structures." In 1957 this was combined with the ACI Standard on buildings to form the more comprehensive current manual of standard practice.

Mr. Delzell was instrumental in the development of a home study course by ACI Committee 315 and the International Correspondence Schools to train technicians in detailing. Based on the ACI standard practices, this course

is an important factor in the dissemination and acceptance of uniform delineation of reinforcement.

Recently, as an active member of the building committee, Mr. Delzell contributed greatly to a successful campaign for contributions toward the financing of the ACI headquarters building.

Wason Medal for Research to Boris Bresler and K. S. Pister

This award goes to Boris Bresler and K. S. Pister for their paper "Strength of Concrete Under Combined Stresses" published in the September, 1958, ACI JOURNAL.

Founded in 1917 by the late Leonard C. Wason, this award is bestowed to an ACI member or members for the most noteworthy research or discovery reported in an Institute paper within a given publication year.

Boris Bresler is a faculty member at the University of California, Berkeley, in the capacity of professor of civil engineering, department of engineering. He graduated from the University of California in 1941, receiving his MS in aeronautical engineering from the California Institute of Technology in 1946.

Following graduation in 1941, he engaged in engineering work with Kaiser Shipyards, Inc., Richmond, Calif., and Consolidated Aircraft Corp., San Diego, through 1945. He then returned to the California Institute of Technology, Pasadena, as a graduate fellow. In 1946 he joined the faculty at the University of California.

Professor Bresler is a member of ACI-ASCE Committee 326, Shear and Diagonal Tension.

K. S. Pister received a BS in civil engineering from the University of California, served in the U. S. Navy Civil Engineer Corps, and in 1946 returned to the University of California for graduate work. Receiving his MS in 1948, he served as lecturer and instructor in theoretical and applied mechanics at the University of Illinois,

B. Bresler

K. S. Pister

while pursuing graduate work in that department. He received his PhD in 1952.

Dr. Pister then rejoined the faculty of the University of California where he has specialized in structural mechanics.

Wason Medal for Most Meritorious Paper to James E. Backstrom, Richard W. Burrows, Harry L. Flack, Richard C. Mielenz, and Vladimir E. Wolkodoff

Five collaborating authors will receive this award for their four-part paper, "Origin, Evolution, and Effects of the Air Void System in Concrete," published in the July, August, September, and October, 1958, JOURNALS.

This award was also established by Mr. Wason in 1917 and is presented for the most meritorious paper by a member or members of the Institute published in the latest volume of ACI *Proceedings*.

James E. Backstrom graduated from Mississippi State College in 1933 and then joined the Bureau of Reclamation where he was assigned to the dam design section. In 1935 he transferred to the bureau's concrete laboratory where he engaged in cement, pozzolan, and concrete investigations for Grand Coulee, Shasta, and Friant dams.

J. E. Blackstrom

From 1939 to 1942 he was employed

R. W. Burrows

H. L. Flack

R. C. Mielenz

V. E. Wolkodoff

in the concrete design section of the Tulsa District, U. S. Army Corps of Engineers, and later on the construction of Canton Dam in Oklahoma. In 1942 he entered the Civil Engineer Corps of the U. S. Navy and served in various capacities in this country and in the South Pacific area until 1946.

He then returned to the Bureau of Reclamation's materials laboratory where he has been engaged primarily in concrete research. He presently is head of the materials, mixes, and durability section of the Concrete Laboratory Branch.

Richard W. Burrows is design consultant for the Martin Co., Denver, contractor for the Titan intercontinental balistic missiles. A graduate of the Colorado School of Mines, he was first employed as a metallurgist by the Aluminum Co. of America, and subsequently served as radar technician in the Navy during World War II.

Mr. Burrows joined the engineering laboratories of the Bureau of Reclamation in 1946 and has written a number of papers on concrete durability. He formulated a chemical grout which was successfully employed in the sealing of leakage at Heart Butte Dam, N. D. He was with the bureau more than 10 years before assuming his present position.

Harry L. Flack was coauthor on Part I of this air void series. Mr. Flack graduated from the University of Colorado and is currently employed as engineer with the Bureau of Reclamation, Denver, engaged in research on the freezing and thawing durability of concrete.

He joined the bureau in 1946 and has conducted various types of investigations and research into the properties of concrete and concrete materials.

Richard C. Mielenz, director of research, The Master Builders Co., Cleveland, is active in ACI committee work and a frequent contributor of technical papers for JOURNAL publication. .

He received his PhD degree in 1939 in geological sciences at the University of California, and began his career as a geologist in 1939 with the Standard Oil Co. of California. In 1941 he joined the Bureau of Reclamation where he remained until 1956 when he made his present affiliation. His main professional activities have been related to selection and testing of aggregates, the microscopy of concrete, concrete admixtures, and the petrography and engineering performance of rocks and soils.

Dr. Mielenz received the ACI Wason Medal for noteworthy research in 1948 for the paper, "Chemical Test for Reactivity Aggregates with Cement Alkalies; Chemical Processes in Cement-Aggregate Reaction," which he coauthored with K. T. Greene and E. J. Benton, published in the November, 1957, JOURNAL. He received the Sanford E. Thompson Award of the American Society for Testing Materials in 1949, 1951, and 1954.

Currently chairman of Committee 116, Research, he is also a member of Committee 212, Admixtures.

Vladimir E. Wolkodoff is senior engineer and head, physical measurements group, research and development division, The Carborundum Co., Niagara Falls, N. Y. His work deals mainly with the investigation of refractories, refractory cements, superrefractories, the manufacturing of synthetic minerals by electro-furnacing, physical properties of abrasives, investigation of extremely hard minerals, and high-temperature microscopy. He has been with The Carborundum Co. since May 1956.

Prior to his present affiliation he was engineering petrographer in the engineering laboratories division of the Bureau of Reclamation, Denver, January 1950 to May 1956. It was during this time that he worked on research material for this four-part series on air void systems.

Mr. Wolkodoff studied at Columbia College and University of Buffalo, receiving his MS degree in mineralogy in 1949. During 1948 and 1949 he was an instructor in mineralogy and crystallography at the University of Buffalo.

Construction Practice Award to J. F. Camellerie

J. F. Camellerie will be cited for his paper, "Slip Form Details and Techniques" published in the April, 1959, JOURNAL. This construction award was founded in 1944 to help enrich the literature of that field of work and to honor the construction man — the man on the job—for his resourcefulness in translating design into a completed structure.

J. F. Camellerie

J. F. Camellerie, vice-president and chief engineer of Plummer Associates, New York, has been active for more than a decade in the design and construction of structures using the slipform method. He is a member of ACI Committee 714, Concrete Bins.

Mr. Camellerie is a 1941 graduate of City College of New York. He joined Plummer Associates (then Wm. Neil Constructors) in 1946.

Presentation of the 1959 ACI awards will be made at the Awards Luncheon on Wednesday, March. 16.

Manuals and Handbooks from ACI

ACI Concrete Primer (1958)

The 1958 edition of the ACI Concrete Primer brings development of the past three decades into the question-and-answer format of this long-popular handbook. Expanded to 72 pages, the revised edition retains its handy pocket size. It develops in simple terms the principles governing concrete mixtures, and shows how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete. Price—$1.00; to ACI Members—$0.50.

ACI Manual of Concrete Inspection (1957)

This 240-page pocket-size book, bound in durable cover, sets up what good practice requires of concrete inspectors and background information on the "why" of such good practice. This fourth edition was revised and extended by Committee 611 to include new practices evolved since the third edition of 1955. Price $3.50—to ACI Members $1.75.

Reinforced Concrete Design Handbook (1955)

An extensive revision by a reconstituted Committee 317 of the original well-known edition of 1939. Tables are revised to reflect changes in range of working stresses encountered in general practice. The original objective to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems is retained. Price $3.50—to ACI Members, $2.00.

New York City, March 14-17

56th Annual ACI Convention

The technical program for the 56th annual ACI meeting in New York City, March 14-17, at the Commodore Hotel, features approximately 30 papers to be presented at eight sessions by eminent concrete authorities. The technical program is geared to the interests of the designer, builder, research man, and others in fields related to the concrete industry.

Monday, March 14, and Tuesday morning, March 15 will be devoted to technical committee meetings.

TECHNICAL PROGRAM

TUESDAY AFTERNOON, MARCH 15

First General Session

Committee Reports:

Committee 326, Shear and Diagonal Tension
Committee 401, Specification for Structural Concrete
Committee 609, Consolidation of Concrete

WEDNESDAY MORNING, MARCH 16

Model Test Session

Structural Models Evaluate Behavior of Concrete Dams—Jerome M. Raphael, University of California, Berkeley.

Glen Canyon Dam Structural Model Tests—George C. Rouse, U. S. Bureau of Reclamation, Denver, Colo.

Experiments with Thin-Shell Structural Models—J. L. Waling and Longin B. Greszczuk, Purdue University, Lafayette, Ind.

Testing Rigid Frame Bridge Model to Ultimate Load—C. S. Wu, California Institute of Technology, Pasadena, and A. A. Pap and D. H. Pletta, Virginia Polytechnic Institute, Blacksburg

Highways Session

Air-Entrained Concrete After 20 Years—E. A. Finney, Michigan State Highway Department, East Lansing.

Continuous Placement of Concrete on Three-Span Continuous Units of Bruckner Expressway—H. B. Britton, State of New York Department of Public Works, Albany.

THE AASHO Road Test—A Background Report—Frank H. Scrivner, AASHO Road Test, Ottawa, Ill.

Gravel Beneficiation in Michigan—F. E. Legg, Jr., University of Michigan, Ann Arbor, and W. W. McLaughlin, Michigan State Highway Department, Lansing

WEDNESDAY AFTERNOON, MARCH 16

Materials Session

Effects of Incomplete Consolidation on Pulse Velocity, Modulus of Elasticity, and Compressive and Flexural Strength of Concrete—M. F. Kaplan, South African Council for Scientific and Industrial Research, Pretoria

Freeze-Thaw Resistance of Concrete Made with Lightweight Aggregates—Paul Klieger, Portland Cement Association, Skokie, Ill.

Effect of Maximum Size of Coarse Aggregates on Properties of Concrete—Stanton Walker and D. L. Bloem,, National Ready Mixed Concrete Association, Washington, D.C.

Concrete-Making Properties of a Portland Blast Furnace Cement Made with High-Magnesia Blast-Furnace Slag—N. Stutterheim, National Building Research Institute, South African Council for Scientific and Industrial Research, Pretoria

Design and Structural Research Session

Ultimate Strength Design—A. L. Parme, Portland Cement Association, Chicago

Effect of Draped Reinforcement on Behavior of Prestressed Concrete Beams—J. G. MacGregor, M. A. Sozen, and C. P. Siess, University of Illinois, Urbana

High Strength Reinforcement in Concrete Structures; Preliminary Report of Committee 339—Robert Sailer, U. S. Bureau of Reclamation, Denver, committee chairman

Shrinkage and Creep of Concrete—Inge Lyse, University of Norway, Trondheim

THURSDAY MORNING, MARCH 17

Restoration of Deteriorated Concrete--Committee 201 Symposium

Conventional Methods of Repairing Concrete—Lewis H. Tuthill, California State Department of Water Resources, Sacramento

Repair of Concrete Pavements—Earl J. Felt, Portland Cement Association, Chicago

The Prepacked Concrete Method—Raymond E. Davis, Consulting Engineer, Berkeley, Calif.

Use of Epoxy Resins—Bailey Tremper, California Division of Highways, Sacramento

Pneumatically Applied Mortar—O. N. Kulberg, Southern California Edison Co., Los Angeles

Design and Construction Session

$1S Million Pier for New York City—David P. Billington, Roberts and Schaefer Co., Inc., New York

Effect of Floor Concrete Strength on Column Strength—Clyde E. Kesler, Robert E. Woods, and Albert C. Bianchini, University of Illinois, Urbana

Welding of Reinforcing Steel Between Precast Concrete Units—J. Neils Thompson, Hudson Matlock, and A. A. Toprac, University of Texas, Austin

Shear Strength of Restrained Concrete Beams Without Web Reinforcement—John E. Bower, U. S. Steel Corp., Monoreville, Pa., and I. M. Viest, AASHO Road Test, Ottawa, Ill.

THURSDAY AFTERNOON, MARCH 17

Research Session--Under auspices of Committee 115, Research

This session differs from general convention sessions in that the information is not released for publication. A group of short papers will be presented on research techniques and preliminary results on a number of current projects not yet sufficiently advanced to justify final reports.

Roger H. Corbetta, president of Corbetta Construction Co., New York, is general chairman of the local committee which is arranging an exhibit to be held in conjunction with the convention, a ladies program, and a number of "extras" to make the 56th annual ACI meeting thoroughly enjoyable as well as profitable.

Price awarded Department of Interior service award

Walter H. Price, past president of the American Concrete Institute, recently received a Distinguished Service Award from the Department of the Interior in recognition of outstanding contributions in the field of engineering with the Bureau of Reclamation, Department of the Interior.

Mr. Price heads the engineering laboratories of the U. S. Bureau of Reclamation, Denver. Since 1951 all laboratories — earth, hydraulic, chemical engineering, and concrete—have been under his guidance.

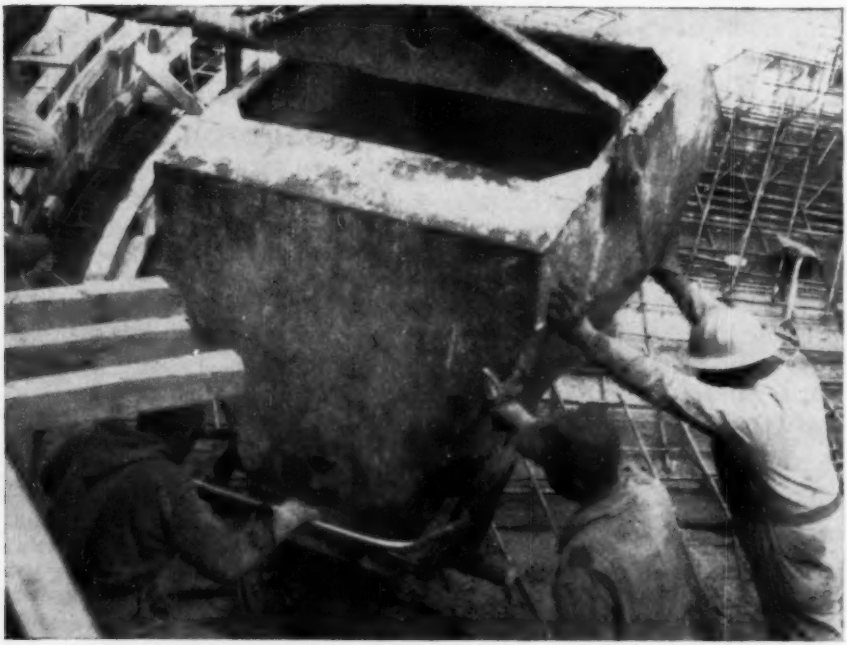
The citation reads in part as follows:

"Since the beginning of his career as a junior engineer, with the Bureau of Reclamation in 1930, Mr. Price has received international recognition as an authority in the field of concrete, and added to the prestige of the Bureau and the Department.

"Under his energetic direction of the concrete and metals laboratory, some of the greatest advances in concrete mix design and construction methods have been realized. Pozzolans became accepted as a money-saving substitute for a portion of the cement in the mass concrete of large dams. Through research under his leadership while in charge of the concrete laboratory, and as Chief of the Engineering Laboratories, the Bureau contributed much to concrete technology, in diagnosing and controlling damaging alkali-aggregate reactions in concrete, in the development of concrete resistant to the attack of sulfate in soils and water, in fundamental studies of air entrainment and the practical methods of using this medium to reduce freezing and thawing damages to concrete, in the application of special admixtures to retard the setting time of concrete and to reduce water and cement requirements . . .

"As a tribute to an eminent career in concrete technology, the Department of the Interior grants to Mr. Price its highest honor, the Distinguished Service Award."

Mr. Price has been the chairman of many committees of ACI, ASTM, and ASCE. He was elected president of the American Concrete Institute in 1957. He was one of the nation's six leading authorities in concrete who inspected Russia's concrete manufacturing plants and concrete construction in 1958. Long active in ACI committee work and author of several JOURNAL papers, Mr. Price received the Institute's Wason Medal in 1951 for his paper, "Factors Influencing Concrete Strength."



No winter slowdown here with Columbia Calcium Chloride in the mix.

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Roads, bridges, buildings . . . no matter what you're constructing, if the jobs use concrete, Columbia Calcium Chloride added at the rate of 2% by weight of cement helps keep your cold weather jobs *moving*. Columbia Calcium Chloride benefits you in these three time-and-profit-saving ways:

Higher Early Strengths—At 40°F, you get 3-day strength in 1 day, 7-day strength in 3½ days. There's no cold weather lag in strength gain; your job stays on schedule.

Faster Initial Set—Under normal conditions, Columbia Calcium Chloride cuts initial set time from 3 hours to 1 hour. At lower temperatures, the effects are even more pronounced. You get finishers on and off the job faster, avoid costly overtime.

Early Form Removal—Rapid strength gain permits removal and re-use of forms in about half the normal time.

Get all these built-in, cold weather benefits for your concrete jobs.

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From the top down —

Roof Slab Concreted First

"SKY HOOKS" were not used, but a Seattle contractor followed the idea in constructing the University of Washington Dr. Samuels Institute Building for Heart and Cancer Control.

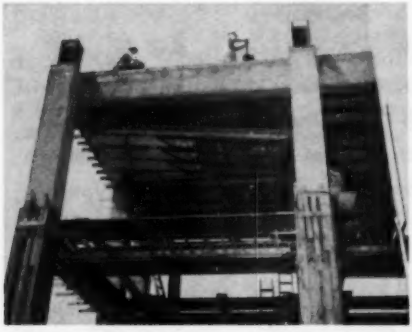
Reversing the usual order of construction, W. G. Clark Construction Co. placed the concrete roof deck first and then worked down from the roof through the third, second, and first floors.

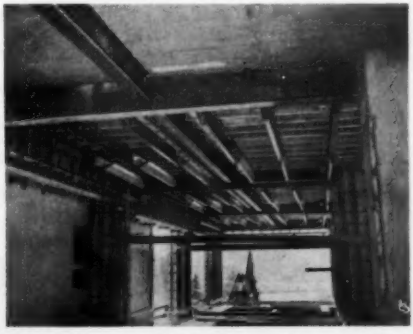
Although designed as a 15-story all steel frame building by Naramore, Bain, Brady & Johnson, Seattle architects, only the first three floors, roof, and penthouse were built at this time.

After erection of the steel frame, the contractor started his concreting with the roof slab. All concrete slabs are 4½ in. thick and the largest panel is 18 ft 4 in. by 10 ft.

The form was made of bolted 4 x 4-in. timbers decked with ¾-in. plywood and supported by three Acrow expandable span units. All the span units were placed lengthwise of the floor panel and were supported at each end on 4 x 4-in. wood wedges that rested on the lower flange of the cross beams of the panel.

After the roof slab had been placed and cured, the supporting assembly, consisting of form and expandable horizontal shoring units, was lowered to the next floor .First the center bolts of the span units were loosened. This allowed the units to sag enough to free the end wedge supports. The entire supporting assembly was then pulled down enough to free the underside of the slab, tipped cornerwise to clear the lower flanges of the panel beams, and then lowered to position on the floor below. After securing the forms, the contractor was ready to concrete the next floor. The entire operation could be completed by four men in about 45 min.





Left—Roof deck forms in place prior to lowering to third floor. (Heavy steel framing is designed for future 15-story building.) Right—Form unit is lowered on completion of roof deck. Four men could lower the panel and position it for concreting on the next floor

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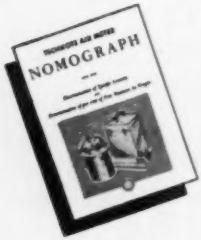
The Techkote Air Meter now provides a multi-range testing instrument that vastly increases the usefulness of the air meter. It is now possible to measure accurately—within a fraction of a degree—entrained air with one instrument on numerous materials such as lightweight concrete, mortar, plaster and soil, in addition to regular concrete.



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Positions and Projects

Southern California chapter and PCA sponsor seminars

Five seminars on the ultimate strength theory of reinforced concrete design were recently conducted jointly by the Southern California chapter of the American Concrete Institute and the Portland Cement Association.

Each of the five seminars consisted of a series of four evening lectures, which were open to architects, engineers, structural designers, and senior engineering students. The seminars were conducted at Van Nuys, San Diego, Long Beach, and Los Angeles during November and December.

ACI Michigan members meet

Engineers from the Michigan area met for an ACI luncheon meeting on November 11.

The subject of the meeting was the design of flat slab floors. This discussion was based on the talk given at the October meeting by Alfred Zweig, senior structural engineer, Albert Kahn Associated Architects and Engineers, Detroit. Discussion centered around limitations in Chapter 10 of the ACI Building Code.

Raymond C. Reese, consulting engineer, Toledo, chairman of Committee 318, Standard Building Code, offered a number of pertinent remarks concerning Chapter 10 in particular and flat slab floors in general. Other comments were made by Mr. Zweig; Daniel Ling, professor of civil engineering, Wayne State University; and William C. Krell, Portland Cement Association, Detroit.

Information concerning flat slab floors contained in "Design of Blast Resistant Construction," by C. S. Whitney, B. G. Anderson, and E. Cohen, published in the March, 1955, ACI JOURNAL, was also covered.

ACI-ASCE committee on composite structures meets

ACI-ASCE Committee 333, Design and Construction of Composite Structures, held a meeting during the ASCE convention in Washington, D.C., Oct. 20, 1959.

The major portion of the discussion was concerned with tentative recommendations for composite designs for buildings. A draft was presented by a task committe headed by P. P. Page, Jr., Seelye, Stevenson, Value and Knecht, New York. The recommendations, explanations, and an introductory statement will constitute the first progress report of the committee.

During the convention, the committee sponsored a technical session on composite design in building construction. Four papers were presented on design, research, and construction.

Ivan M. Viest, AASHO Road Test, Ottawa, Ill., is chairman of the joint committee.

Rice addresses ACI Southern California Chapter

Edward K. Rice of T. Y. Lin and Associates, Van Nuys, Calif., was the feature speaker at the ACI Southern California Chapter meeting on November 18. He presented an illustrated talk on "Recent Developments in Prestressed Concrete over the United States."

J. L. Peterson (left), chapter president, and Edward K. Rice, guest speaker

Over 125 members and guests attended the dinner meeting.

The March meeting will be the annual meeting of the chapter. New chapter officers elected by ballot will be announced at this meeting.

Battelle reports new analysis technique

A new technique, using radioisotopes, to speed up the analysis of magnesium oxide in portland cement has recently been reported by scientists at Battelle Memorial Institute, Columbus, Ohio. The radiometric technique, developed in a study of tracer quality-control systems for the U. S. Atomic Energy Commission, cuts the time required for analysis of magnesium oxide from 28 hr to less than 1 hr.

The new procedure for analysis is based on modification of the basic method outlined by the American Society of Testing Materials. The primary change in the latter is the labeling of ammonium monohydrogen phosphate with phosphorus-32. This step enables direct determination of the amount of precipitate formed and eliminates the time-consuming digestion, filtration, ignition, and weighing steps required in conventional analysis. In a concentration range of 5 to 15 milligrams of magnesium oxide per 100 milliliters, accuracy of the results have been within 0.05 percent when the results are expressed as percent of magneisum oxide in cement.

International research experts address BRI fall conferences

Distinguished building researchers representing the governments of Canada and South Africa presented papers at the "Building Research, International" session of the Building Research Institute's fall meeting held November 17-19 in Washington, D. C. at the Shoreham Hotel. Recent research developments in Scandinavia, Russia, and Holland were outlined by BRI members who attended this summer's International Building Conference (CIB) in Rotterdam.

Prominent ACI members participating in the conference were: Douglas E. Parsons, chief, Building Technology Division, National Bureau of Standards, who presided at this session, November 17; M. F. Kaplan of Pretoria, head of the Civil Engineering Division, Building Research Institute of South Africa, who outlined the work of that organization, much of whose literature is distributed in America; and Grayson Gill, engineer-architect, Dallas, who was one of the speakers reporting on the Rotterdam meeting.

Calaveras to merge with Flintkote

Calaveras Cement Co. will be merged into Flintkote Co. in a move described as a further step in Flintkote's planned program of expansion and diversification.

Calaveras, based in San Francisco, owns and operates a cement plant in San Andreas, Calif., with an annual capacity of 4,300,000 bbl. All employees and executive personnel of Calaveras will be retained and Wallace M. Mein, Calaveras chairman of the board, will become a Flintkote director.

Neu appointed to manage Garavaglia Contractors

Frederick W. Neu was recently appointed vice-president and general manager of Louis Garavaglia Contractors, Inc., Detroit, Mich.

Mr. Neu leaves a top management post with the firm of Grove, Shepherd, Wilson and Kruge with principal offices in New York City. He has been employed by this company for the past 20 years.

Mr. Neu is a graduate civil engineer and is registered as a civil engineer in Michigan and Indiana. His experience includes various sorts of heavy construction including bridges, buildings, foundations, and highways, which is the line of work followed by Garavaglia Contractors.

Harry F. Thomson

Harry F. Thomson, past president of the Institute, died on November 7 in Mexico City.

After graduating from Washington University, Mr. Thomson served as instructor and research worker at the Massachusetts Institute of Technology for 4 years. He then became associated with the Provident Chemical Works, successively as assistant superintendent, Eastern manager in New York City, sales manager, and vice-president. In 1924 he joined the Federal Phosphorous Co. as executive assistant.

In 1927 Mr. Thomson became vice-president of the General Material Corp. of St. Louis, one of the pioneer companies in producing commerical ready-mixed concrete. He continued with that company as vice-president and subsequently as president until 1948. Since that time he had acted as consulting engineer, specializing in ready-mixed concrete plant and merchandising problems.

Mr. Thomson had been active in ACI affairs since 1928. In 1951 he was elected president, culminating years of participation in various Institute activities. He was elected to the Institute Board of Direction in 1939 and served until 1940, then again from 1942-48, and was elected vice-president in 1949. He was active in the work of numerous committees and contributed generously to JOURNAL pages.

Mr. Thomson served as president of the National Ready Mixed Concrete Association and was a former national director of ASCE.

Long active in the civic affairs of St. Louis, he had served as director of the St. Louis Chamber of Commerce and was one of the organizers of the Metropolitan Plan Association of St. Louis. He had cooperated actively with the federal emergency and wartime agencies; these services included membership on the Advisory Committee, Office of Defense Transportation for the St. Louis region; chairmanship of the

National Advisory Committee for the Ready-Mixed Concrete Industry, Office of Price Administration; and panel member for industry of the Regional War Labor Board.

Mr. Thomson was a retired lieutenant commander of the Civil Engineering Corps. Reserve of the U. S. Navy.

Besser establishes technical center

Establishment of Besser Technical Center dedicated to the advancement of the concrete masonry industry has been announced by J. H. Besser, president of Besser Co., Alpena, Mich.

The technical center has three departments each serving blockmakers in specific ways: Research, Materials and Methods, and Consulting Service. The aim of the center is to exhaustively explore these fields and inform the industry through correspondence and formal instruction. Attendance at the Besser School is free and services of the center are available to all interested in the masonry field.

Since 1954 the Besser Co. has conducted the Besser School for Blockmakers to inform the operators of the best known production techniques. In

1958 a Seminar for Block Plant Owners and Managers was added to the school curriculum. By the end of 1960 the service of the school will be further enlarged with instruction in the operation of automatic off bearing and unloading machinery.

National prestressed concrete short course, February 1-3

The third national prestressed concrete short course cosponsored by the department of civil engineering, University of Florida, and the Prestressed Concrete Institute, will be conducted February 1-3 at the University of Florida, Gainesville.

The program is planned to acquaint the practicing engineer and architect of developments in theory and design of prestressed concrete structures. It will enable the producers of prestressed concrete units to learn efficient planning and production in casting yards. Outstanding engineers from all over the country will discuss topics on specifications, quality control and design of bridges, industrial buildings and unusual structures.

Further details available from: Prestressed Concrete Short Course, Civil Engineering Department, University of Florida, Gainesville, Fla.

Rawhauser retires from government service

Clarence Rawhauser has retired from government employ after 30 years of continuous service.

For the past 7½ years he has been with the International Cooperation Administration of the U. S. State Department: 7 years on assignment as dams design engineer adviser with the Central Water and Power Commission of the Government of India, and 6 months on the Washington complement of ICA in the industrial engineering division. While on assignment in India he worked closely with design supervisors and construction engineers on many of the large concrete and masonry dams under construction including Bhakra, Airakud, Rihand, and Koyna dams.

For over 21 years, prior to the Indian assignment, Mr. Rawhauser was a designer in the Concrete Dams Division of the Bureau of Reclamation at Denver. He contributed materially to the plans and methods used for controlling the temperature of Seminole, Bartlett, Parker, Grand Coulee, Marshall Ford, Shasta, Friant, and Altus dams.

He has been an ACI member since 1937 and was awarded the Wason Medal for the most meritorious paper in 1945, "Cracking and Temperature Control of Concrete Dams," published in the February, 1945, JOURNAL.

Mr. Rawhouser, tentatively located in Seattle, plans to work as a consultant on dams and development of river valley project plans involving flood control, power, irrigation, and navigation.

Albright joins PCA staff

Richard O. Albright has joined the Portland Cement Association staff as a field engineer. He will represent PCA in northwestern Indiana.

For the past 5 years Mr. Albright has been a designer for Clyde E. Williams and Associates of South Bend, Ind.

Errata

The following corrections should be made in "Design of L-Shaped Columns with Small Eccentricities," by L. S. Muller, which appeared in the December, 1959, ACI JOURNAL.

p. 489—in the notation, in the expression for $r_{g\,min}$ change A_s to A_g. In the expression for X_s change b to B.

p. 494—in the definitions above Eq. (20), the term $F_b = 0.45f_c'$ should be a separate item and not part of the expression for f'_{bg}. The term μ_s under "Concrete section," should read $\mu_s = -0.0122$.

p. 496—the correct title in Reference 4 is "Spannungnachweise bei schiefer Biegung mit und ohne Laugskraft in belieiebigen Querschnitten."

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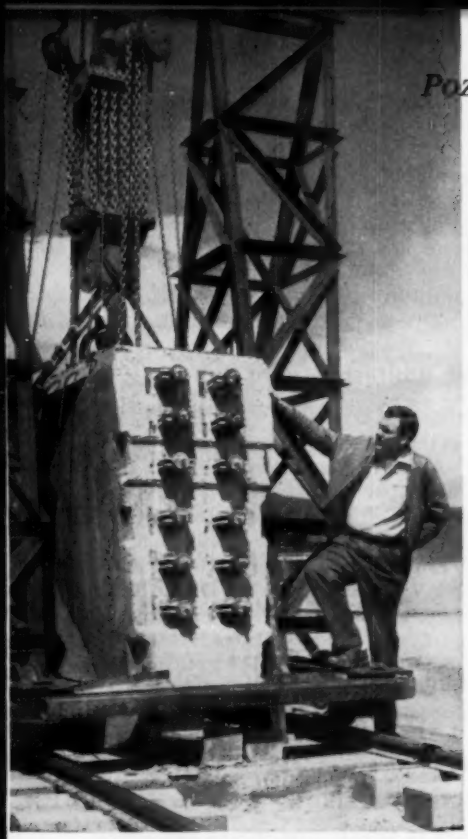
Prestressed concrete beams for 6 bridges at Air Academy



PRECAST BRIDGE BEAM is inspected after post-tensioning. Construction of all prestressed POZZOLITH concrete beams by A. S. Horner Construction Co., Contractors, Denver. Bridges built under supervision of the Air Force Academy Construction Agency.

Part of the striking man-made landscape at the new U. S. Air Force Academy are six major concrete bridges—4 highway and 2 railway spans—that required over 2½ miles of prestressed concrete beams.

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Medusa announces executive appointments

The Medusa Portland Cement Co., Cleveland, Ohio, has appointed Ralph H. Campbell, formerly vice-president in charge of sales, as vice-president-commercial. Robert A. Marshall, formerly assistant general sales manager, has been appointed general sales manager.

Mr. Campbell has been associated with Medusa for over 25 years. Mr. Marshall has been with the company 6 years.

FIP council meets in Madrid

The administrative council of the Federation Internationale de la Precontrainte recently met at the Instituto Tecnico de la Construccion y del Cemento in Madrid.

It was agreed that the 4th Congress of the FIP would be held in Rome or Naples in June or July 1962. The program will be based on research construction, significant developments, and events of importance in the field since January 1958. The six official FIP languages will be English, French, German, Italian, Russian, and Spanish.

Ford, Bacon and Davis granted Australian project contract

Ford, Bacon and Davis, Inc., New York engineering firm, has been signed by the Queensland government to supervise the first phase of a $22.5 million rehabilitation and modernization railway program.

The immediate program involves work on the Townsville-Mount Isa railway line, including a new classification yard near Townsville; relaying with heavy rail the 242 miles between Richmond and Duchess on the western portion of the Mount Isa line, and restoring some 300 timber bridges in this area; also reducing grades and improving drainage.

Ford, Bacon and Davis will advise the project unit and provide over-all supervision of engineering, design, and construction. D. G. Niarchose, senior civil engineer, has been named resident engineer and L. H. Poor, senior supervising engineer, will represent the firm as consulting engineer on the project, making periodic trips to Australia.

Viest addresses joint ASCE-ICE group in Toronto

Ivan M. Viest, bridge research engineer, AASHO Road Test, Ottawa, Ill., was principal speaker at a joint area meeting of the American Society of Civil Engineers and the Institution of Civil Engineers in Toronto, November 10. Dr. Viest, chairman of ACI-ASCE Committee 333, Design and Construction of Composite Structures, reviewed the experimental and theoretical investigations and significant applications of composite construction.

Short course in residential construction, January 14-15

Ways in which good design and construction can be incorporated into houses built for low income groups will be explained at the 15th annual short course in residential construction, Jan. 14–15, 1960, at the University of Illinois.

The short course is the annual report session of the University of Illinois Small Homes Council-Building Research Council. One of the subjects to be covered is slab construction.

Permanente announces major executive appointments

Peter S. Hass was recently elected executive vice-president of Permanente Cement Co., Oakland, Calif. Mr. Hass joined the company in 1947 as senior engineer at their plant near San Jose. He subsequently held a number of administrative positions and in 1955 was elected vice-president and assistant general manager of the company.

Three major appointments have been made in the operations division: John I. Walker, formerly assistant to John

M. Groutte, vice-president in charge of operations has been assigned to the newly created post of general production manager. Walter E. Ousterman, previously a staff assistant, has been named assistant to the vice-president of operations. Eldred Bollinger, formerly assistant quality control manager, assumes the position of assistant production manager. All three appointees are headquartered at the Permanente plant near Los Altos, Calif.

Sprouse to manage new Master Builders Oklahoma City branch

James H. Sprouse, Jr. will serve as office manager for The Master Builders Co. new branch office in Oklahoma City.

Mr. Sprouse joined Master Builders in 1955. Prior to his new appointment he was a Master Builders field sales engineer in Texas, Oklahoma, and New Mexico.

AASHO road test chalks up five million miles

Vehicles at the AASHO Road Test, Ottawa, Ill., have chalked up 5,000,000 miles of operation in the first year of the full-scale test traffic. This pace has resulted in an average of 325,000 axle load applications on each of the hundreds of test pavement sections.

The Road Test, sponsored by the American Association of State Highway Officials, is a study of the performance of pavements and short-span bridges of various structural designs under controlled truck traffic. Present plans call for test traffic to continue until the summer of 1960.

Three main experiments are under way at the test site. They involve rigid pavements, flexible pavements, and short-span bridges. In the pavement experiments some test sections were underdesigned and some were overdesigned. However, each experiment is

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independent and requires separate analysis. All of the 18 bridge spans included in the test were underdesigned. Each span is considered a separate case study.

Ten vehicles range from pick-up trucks with 2000 lb single axle loads up to large tractor and semitrailer combinations with 30,000 lb single and 48,000 lb tandem axle loads. Traffic operates on five test loops in ten lanes. All vehicles in any one lane have identical axle loads.

The test vehicles are being driven by men from a special U. S. Army Transportation Corps unit stationed at the test site. The soldiers drive in two shifts, covering about 18½ hr per day, 6 days a week.

No data or detailed information on the performance of the various pavements or bridges has been released. The Highway Research Board, according to staff officials, is convinced that release of data prior to completion of the tests might lead to misleading conclusions.

Two reports on the test are being prepared and are scheduled for publication by the Board next year. The first will cover the background and concepts of the test; the second will describe the construction of the test facilities. Final reports on the research findings will be published in 1961.

Sika opens district office in Kansas City

Sika Chemical Corp. has opened a district office in Kansas City, Mo. This is its tenth district office and will serve architects, engineers, contractors and the concrete construction industry in Missouri and Kansas.

John F. Esping heads the new district office. For the past 8 years Mr. Esping has been a district manager and sales representative in the Kansas City area for Butler Manufacturing Co., Reynolds Metals Co., and the Special Products Division of Stromberg-Carlson Co.

Burnett elected president of Johns-Manville

C. B. Burnett has been elected president and chief operating officer of Johns–Manville Corp., New York. Mr. Burnett joined Johns-Manville in 1931. He was elected executive vice-president and a director in 1957.

Twaits-Wittenberg Appoint Ball to key post

The appointment of Herbert C. Ball to the position of general manager of Twaits-Wittenberg Co., Los Angeles constructors and engineers, was recently announced by Carl H. Wittenberg, general partner. Mr. Ball joined the Twaits-Wittenberg organization in 1934. In 1958 he was made construction manager, a position he held until his present appointment.

Alpha Portland Cement registers new trademark

Alpha Portland Cement Co., Easton, Pa., has registered a new trademark with the U. S. Patent Office.

Alpha now has a capacity of over 14,000,000 bbl a year and the modernized trademark symbolizes Alpha's progressiveness according to the company president, Robert S. Gerstell. It is the second trademark registered by Alpha since the company was organized in 1895.

University of Houston sponsors short course

A short course covering prestressed concrete design was held at the University of Houston, November 10 to December 15, on Tuesday evenings.

The course was conducted by Ben W. Young, Prescon Corp., Corpus Christi, Tex. The program was designed to provide engineers with a working knowledge of prestressed concrete design and to develop for each engineer a notebook containing typical examples for subsequent reference.

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Neff Concrete appoints Schmidt superintendent

Crosby A. Schmidt has been named plant superintendent at Neff Concrete Products Co., Danville, Ill. Mr. Schmidt previously was vice-president and general manager of Prestcrete Corp., Plano, Ill.

Stone and Webster promote Allshouse

Robert L. Allshouse, superintendent of construction for Stone and Webster Engineering Corp. in Texas and Louisiana, has been named a construction manager of the world-wide organization and transferred to the Boston headquarters. Mr. Allshouse joined the company in 1952.

King joins Master Builders

John C. King has been named product development engineer for The Master Builders Co., Cleveland. Prior to joining Master Builders, Mr. King was chief engineer for Intrusion-Prepakt, Inc. of Cleveland. He has also been associated with International Engineering Co., Denver, and the Bureau of Reclamation where he specialized in dam design and mass concrete research.

Calaveras announces plans for $14 million cement plant

Calaveras Cement Co., a division of The Flintkote Co., with headquarters in San Francisco, recently announced plans to construct a $14 million cement plant at Redding, Calif. Construction will begin shortly and is scheduled for completion by March, 1961.

The new plant, to be situated on a raw material site encompassing more than 1500 acres of high grade limestone and shale, will have an initial rated annual capacity of 1,500,00 bbl. The new plant will help meet the demands in adjacent areas now being served by the San Andreas plant of Calaveras Cement Co.

Honor Roll

January 1 —— December 31, 1959

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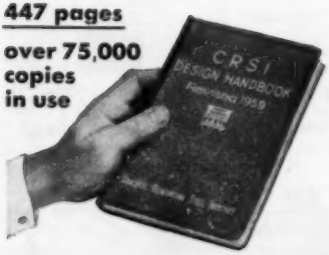

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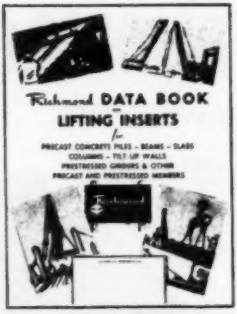

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Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

THE PRESCON SYSTEM—Folder explains the advantages of prestressed cast-in-place concrete tensioned by the Prescon System of post-tensioning in modern construction. Various examples of buildings under construction and detailing of tendon and transverse slab steel in place on a job using plywood forms are shown.—Prescon Corp., Corpus Christi, Tex.

VOIDCRETE (Catalog 601)—Complete catalog on Elgood reusable inflatable rubber void forms for prestressed and precast products; Cast-in-place cored roof and floor slabs, bridge decks, columns, piers; drains; and monolithic sewers and pressure pipe.—Elgood Concrete Forms Corp., 378 Ten Eyck St., Brooklyn, N.Y.

NONDESTRUCTIVE TESTING HANDBOOK—This handbook integrates the experience acquired throughout industry to date in selecting, applying, and interpreting tests designed to improve performance reliability of equipment, to cut costs, to insure product quality or uniformity, to increase the utilization of materials, and reduce breakdowns and accidents. Edited by Robert C. McMaster of The Ohio State University with a board of 106 contributing and consulting editors selected from this country and abroad. Organized in 54 coordinated sections, the handbook offers hard-to-find guidance to those who conduct tests, prepare reports, and maintain records. Over 1200 charts, drawings, forms, and photographs are included. $24 in 2 volumes.—The Ronald Press Co., 15 East 26th St., New York 10, N.Y.

NSA SLAG BLOCK MANUAL—Prepared by the National Slag Association to be of assistance to architects, builders, specification writers, and manufacturers of masonry units. Provides specific information on the properties of slag and the resultant masonry units; discusses acoustical properties. Tables furnish fire resistance and fire endurance ratings.—National Slag Association, 613 Perpetual Bldg., Washington 4, D.C.

FROM MOUNTAINS TO MICRONS—Color, 16-mm sound motion picture (25 min), "From Mountains to Microns" graphically details the portland process of creating cement. Shows how jet runways, rocket launching pads, highways, skyscrapers, bridges, and dams are made possible through the manufacture and manipulation of cement. Avail-

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able on free loan to interested adult groups, high school and college classes throughout the country through courtesy of the Portland Cement Association.—Modern Talking Picture Service, 3 East 54th St., New York, N.Y.

TECHNICAL SUPPLEMENT TO "THE TESTING WORLD"—Soiltest, Inc. has published a supplement featuring "The Story Behind the New Standard Sieve Specifications." The article covers sieve specifications from the first fixed ratio sieve series in 1910 to the recent publication by the American Society for Testing Materials of revised specifications for standard testing sieves, ASTM E-11-58.—Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

PATENTS AND INVENTIONS—AN INFORMATION AID FOR INVENTORS—Pamphlet published by the U. S. Patent Office contains basic information as to patenting and patent procedures, and related information which may prove helpful in the development and marketing of inventions. Single copy, 15 cents.—Superintendent of Documents, U. S. Government Printing Office, Washington, D.C.

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EMERI-EPOX (Bulletin EX-1) — Properties, recommended applications, and special advantages of Emeri-Epox, a floor surfacing and patching material combining emery and epoxy resins are set forth in 2-page bulletin. Choice of available colors is listed.—Walter Maguire Co., Inc., 60 East 42nd St., New York 17, N.Y.

SCI CATALOG (No. 59A)—Contains illustrations and descriptions of 73 pieces of equipment manufactured as standard products by Custom Scientific Instruments, Inc. for production, control, and research.—Custom Scientific Instruments, Inc., Kearny, N.J.

LOOKING AHEAD

Jan. 18-20, 1960 — 11th Annual Meeting, National Concrete Products Association (Canada), Royal Alexandria Hotel, Winnipeg, Ontario, Canada.

Jan. 18-21, 1960 — 58th Annual Convention, American Road Builders Association, Netherland Hilton Hotel, Cincinnati, Ohio.

Jan. 18-21, 1960 — Annual Convention, National Limestone Institute, Statler-Hilton Hotel, Washington, D. C.

Jan. 24-28, 1960 — 41st Annual Meeting, Associated Equipment Distributors, Conrad Hilton Hotel, Chicago, Ill.

Jan. 25-28, 1960 — 11th Annual Plant Maintenance and Engineering Conference and Exhibit, Convention Hall, Philadelphia.

Jan. 25-29, 1960—Stress Measurement Symposium, Arizona State University, Tempe, Ariz.

Feb. 1-5, 1960 — Committee Week, American Society for Testing Materials, Hotel Sherman, Chicago, Ill.

Feb. 15-19, 1960 — Annual Convention and Exposition, National Sand and Gravel Association, Conrad Hilton Hotel and Coliseum, Chicago, Ill.

Feb. 22-24, 1960 — Annual Convention and Exposition, National Crushed Stone Association, Conrad Hilton Hotel, Chicago, Ill.

Mar. 7-10, 1960 — 52nd Annual Convention, American Concrete Pipe Association, Fairmont Hotel, San Francisco, Calif.

Mar. 7-11, 1960 — New Orleans Convention, American Society of Civil Engineers, Jung Hotel, New Orleans, La.

Mar. 14-17, 1960 — 56th Annual Meeting, American Concrete Institute, Commodore Hotel, New York, N. Y.

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